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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

CHANNELIZATION OF MOTOR TRAFFIC

BY GUY KELCEY,[1] M. AM. SOC. C. E.

SYNOPSIS

The motor car provides great individual freedom, range, and flexibility of movement. Many millions of these motor vehicles are operated, at high speeds, along a multiplicity of intersecting roadways. Although most drivers, both amateur and professional, have acquired surprising skill, traffic accidents and congestion have become a major public problem.

Channelization, which refers to means by which certain aspects of this problem may be attacked, proceeds on the principle that traffic safety and efficient roadway use are each necessary to, and functions of, the other and that both are products of order. By confining movements of vehicles to definite lanes, channelization avoids or reduces disorders arising from conflicting movements. In part, too, it accomplishes its purposes by simplifications to which drivers respond naturally. It utilizes the skill and ability of drivers as a whole and, to a considerable extent, prevents or neutralizes faulty driving. The result is a self-regulated flow which insures orderly operation of vehicles by drivers as individuals and by groups.

CHANNELIZATION

The history of civilization is a record of transport which, in its turn, is a story of channels. Making use of many kinds of transport for himself and his possessions, man traveled and "took root," through the centuries, mainly over water and along natural land channels, which followed the most convenient courses between major, distant points. Caravan routes, the Santa Fe and Oregon trails, Mississippi River traffic, and even modern air routes are examples.

Man settled the narrow strips of good land adjacent to these routes of travel. At major junctions, bulges of population developed into the important cities of today. These over-crowded peaks of population are a consequence of

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **April 15, 1940.**

[1] Sales Mgr., Signal Service Corporation, Elizabeth, N. J.

earlier forms of transport, which provided conveyance between major points but did not do a good job of local distribution. The motor car, which is distributing these bulges of population more uniformly and over wide areas, provides greater use of existing channels, is creating a multiplicity of new ones, and is reaching others not hitherto accesible. It has multiplied, by many times, man's speeds, his range, and the number of courses along which he may travel and adjacent to which he may settle.

All important transport, even on wide oceans and in the three-dimensional air, operates, then, within the limits of quite definite and very narrow paths. As these paths become crowded, or because they are not exactly fixed, those who use them must conform to quite exact rules. In particular, these rules are made to keep travel within the limits of directional or one-way channels so that opposing and cross movements shall be in least interference. Any failure to adhere to these rules and to keep within assigned channel limits reduces flow efficiency and increases hazard. The establishment and strict application of such rules has not proceeded as far with respect to the operation of the motor car as it has with relation to other forms of transport.

As a newcomer in the transport system, the motor car has, in a sense, shouldered its way into the general scheme of things. It is to be expected that its intrusion into, and its impact on, existing institutions, practices, and habits should have many dislocating and jarring effects. On the other hand, it is the only means of power transport available to the non-professional and individual operator generally, and these amateur drivers constitute the great majority of automobile operators. It is amazing how competent many of them are, what a slight degree of control they actually need, and what a complete and easily controlled device the motor car has become. In fact a survey of accidents involving drivers in Connecticut (particularly accident repeaters), for a period of six years, discloses 98.7% of all drivers to be normally efficient.[2]

The surprising fact, then, is not the congestion, the accidents and other problems which the motor vehicle has introduced, but that so much progress has been made in so short a time in the evolution of such a vast, new and vital means of transport without the development of problems that are much worse.

There are so many automobiles, and they move so fast that it has not been possible, by police or other regulation, to force all drivers to conform exactly to the rules that are aimed to insure orderly movement. Neither has it been possible to educate or force each driver to keep in his own place or channel and to keep out of wrong ones because most roadways permit and often invite wrong movements. It is the writer's contention that if proper directional channels are made logical, easy, and inviting to use, and if others are made unattractive and difficult, drivers (who, in the main, are surprisingly competent anyway) will follow lines of least resistance and use the right ones. Thus quite a number of the traffic problems that bother traffic officials will cease to exist for lack of cause. The subject of this paper—channelization—refers, then, to the means by which good driving is made easy and bad driving difficult.

[2] "The Normal Automobile Driver as a Traffic Problem," by T. W. Forbes (of the Bureau of Street Traffic Research, Yale University, New Haven, Conn.), *Journal of General Psychology*, 1939, pp. 471–474.

Of course, the conception of one side of the road being used for cars traveling in one direction and the other side for cars going the other, even if the road is not physically divided, is elemental. This rule "Keep to the Right" (or left, in some countries) is so thoroughly comprehended by all persons that it constitutes a foundation upon which the solution of any traffic problem must be based.

It is interesting to report that Mr. William Phelps Eno[3] defined and adapted many of the principles of channelization early in this century by applying them to the solution of problems introduced by increasing concentrations of carriages, bicycles, and early motor cars in New York, N. Y., Washington, D. C., and in Europe. The creed of the Eno Foundation for Highway Traffic Control, "Ex chao ordo" (out of chaos, order) is a wholly satisfactory introduction to the writer's subject, and to the statement that efficient, smooth-flowing, safe, motor traffic is a product and consequence of order. Channelization refers to means by which traffic may be made self-regulating and orderly.

The motorist is often charged with errors which, in reality, are natural consequences of disorder arising from inadequate or faulty roadway conditions that invite such errors. Fluid traffic flow and safety, which are functions each of the other, are both products of order; and order, in turn, is one function of proper roadway design.

Channelization, which is concerned primarily with problems of the pavement surface and above the surface, establishes proper lanes within which movements of vehicles are confined. It increases the utility and safety of automobile transport by simplifying traffic flow and by eliminating confusions, conflicts, and eddy currents. It takes advantage of the high degree of skill and instinctive driving capacity possessed by a vast majority of drivers and does much to neutralize the unsatisfactory performance of a few drivers who may not drive properly, either occasionally or habitually. Finally, it invites and encourages good driving manners, bad ones being quite often more an evidence of confusion or desperation than of poor sportsmanship.

Human Factors

In seeking causes of traffic difficulties and in applying corrections, several factors, the more important of which are mentioned herewith, must be kept in mind.

Instinct Factors.—Many driving practices, including most good ones, naturally become habits, which, in turn, become instinctive. Timing and coordination, which are instinctive factors, are as necessary to good driving as to good music. In working with traffic, it is important to establish conditions favorable to the natural exercise of these instinctive qualities so that the driver's mind may be on the alert, and free to "captain" the vehicle, to navigate, and to attend to small adjustments and driving emergencies.

Course of Least Resistance, Speed, and Simplicity.—A driver tends to take the most convenient course, the course which calls for the least effort, causes

[3] "Street Traffic Regulation," 1909, p. 7, Sections 437 and 440, pp. 10, 11, 28, 29, and "The Science of Highway Traffic Regulation," 1920, pp. 26–39, Eno Foundation for Highway Traffic Control, Saugatuck, Conn.

least discomfort, and requires least reduction of speed. Channelization, which blocks wrong courses or makes them difficult, makes correct ones easy, convenient, and inviting.

Traffic solutions should be based on the fact that drivers tend to operate at speeds up to the limit of their feeling of security and comfort, but few drivers press beyond this limit. Although this speed factor varies, depending on the driver, the car, the weather, and other conditions, approximate speeds at which normally good drivers (and this means most of the traffic load) will operate, under a given set of circumstances, usually can be predicted or arranged in advance.

Advertisers tell it to the "Zilches" and the "Stuyvesants" understand. Channelization, which is designed to be understood by the few who are poor drivers, offers no difficulties to the great majority who are good ones.

Heads Up.—When boys walk railroad rails those who keep their heads up go farthest. A structural worker on "high steel" fixes his attention ahead. To walk a line one must look at a point that is up and ahead.

A good driver must keep his head and his attention up. He does not, then, so much approach objects 100, 200, 300 or more feet in front (depending on speed and circumstances) as pass them. Thus, while his attention comprehends both near and far range of roadway in front, his major interest is focused well ahead, observing and evolving his plan of action and checking it constantly. Except in the case of necessity, he gives near ranges checking and glance attention only. The more open the road and the greater his speed, the farther ahead is his attention focused.

Roadway design should, and channelization treatments must, be laid out with this factor in mind. What the driver is to do must unfold and be sufficiently apparent from a distance so that his major attention may be given to other matters on closer approach, and so that he may continue to operate his vehicle with his head up. Since channelization eliminates irregular vehicular movements, a driver's attention is not distracted by the treatment itself or by the possibility of surprises from other vehicles. Therefore, he is free to form, and to execute, good judgments with respect to the fewer and lesser problems that remain, both immediately in front and far ahead of him.

Since channelization takes advantage of instinctive driving propensities, the success of a channelizing treatment will usually be in proportion to its glance or instinct value. Such a treatment must have no surprises or "ambushes" and must be so well thought out and designed that it need not be solved consciously by the driver. A driver goes in only one direction at a time and, progressively, should make only one decision at a time. If the treatment is well done, he is never in doubt because his course and his next action will always have a one-step-at-a-time simplicity.

Driving Factors

Merging, Intersecting, and Opposing Movements.—In the illustrations that follow, arrows indicate courses and directions of vehicles. However, arrows cannot represent, accurately, the effect of a moving vehicle as a zone of in-

fluence that varies with velocity and is subject, as well, to driver characteristics and other factors. Thus, a standing car influences only its area. When this car moves, it commands leeway on both sides and clearance ahead which, at speed, is many car lengths in extent. The intersection of the courses of vehicles, then, involves moving zones of influence with the actual vehicle approximately centered, as a core, in the rear of each zone.

When two cars operate in the same direction, the movement is a merging one and the relative speed of the merger approaches the difference of their speeds, as a limit. Thus cars *A* and *B* (Fig. 1), moving parallel to each other at 45 miles per hr and 40 miles per hr, have a relative speed of 5 miles per hr.

When two cars, *A* and *E* (Fig. 2), intersect at a right angle, each maneuvers with respect to the other at his own speed. Each driver works to a definite

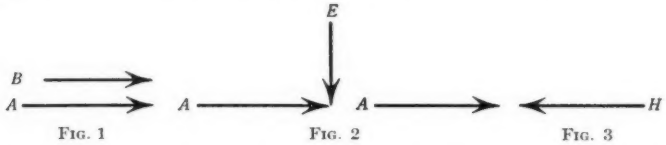

FIG. 1 FIG. 2 FIG. 3

reference point and, subject to visibility and high-speed factors, each gages the actions of the other with reasonable accuracy because, relatively, each moves across, rather than along, the other's range of vision.

Two cars, *A* and *H* (Fig. 3), moving in opposite directions at 45 miles per hr and 40 miles per hr, have a relative speed which is the sum of their speeds, or 85 miles per hr. The exercise of good judgment in the face of high, relative, head-on, speed is often difficult, where the courses of vehicles intersect, because each driver moves along (as distinguished from across) the other's range of vision and neither has any definite reference point to guide to.

The movement of two cars in the same direction and parallel to each other is most efficient and the safest. Progressively (see Fig. 4), as the movement of

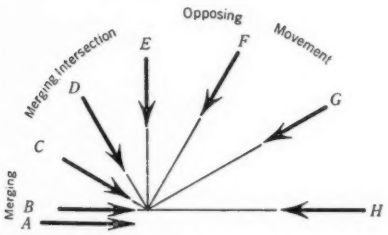

FIG. 4.—SPEEDS OF *B*, *C*, *D*, *E*, *F*, *G*, AND *H* MOVING AT 40 MILES PER HR RELATIVE TO *A* AT 45 MILES PER HR

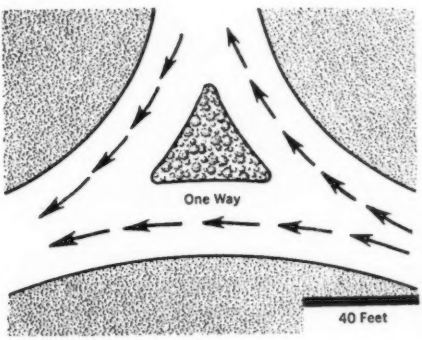

FIG. 5

cars becomes increasingly opposed (as *A* and *B*, *A* and *C*, *A* and *D*, *A* and *E*, *A* and *F*, *A* and *G*, and *A* and *H*) conditions favorable to natural or instinctive driving decline. Uncertainty and hazard increase and efficiency of movement is reduced. The increasing relative speed of car *A*, at 45 miles per hr, with

respect to *B*, *C*, *D*, *E*, *F*, *G*, and *H*, at 40 miles per hr, is indicated by scaled lines in Fig. 4. This increase in relative speed ranges from the difference of speeds, as in the case of *A* and *B*, to the sum of the speeds, in the case of *A* and *H*.

As traffic movements become increasingly opposed, confusion and uncertainties develop and multiply. Under more complicated circumstances, involving fixed objects and many moving vehicles from various directions, in combination with a large number of possible paths, quite complex situations of rapidly changing relationships often upset the natural rhythm of driver actions. Resulting disorders are progressive and, once started, are most difficult to control. Channelization accomplishes the easier objective of preventing the introduction of disorders.

A traffic circle (Fig. 5) with a diameter, of several hundred feet, would provide a merging movement of vehicles which would be essentially parallel. Traffic flow would be continuous, and maneuvering of vehicles with respect to each other would be at the difference of their speeds.

If the diameter of such a circle is reduced gradually, the courses of vehicles will merge and finally cross at steeper and steeper angles which approach a right-angle intersection (as in Fig. 6) as a limit.

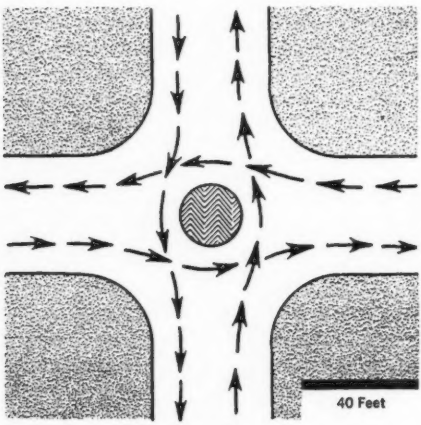

FIG. 6

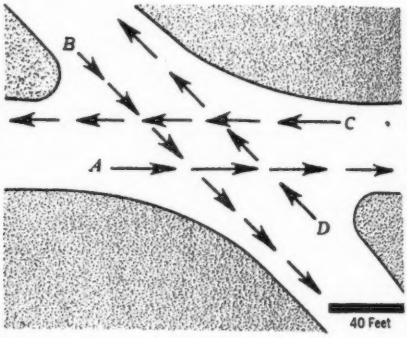

FIG. 7

To go beyond a right-angle intersection creates opposing movements which should be avoided. In Fig. 7, path *A* merges with path *B* but opposes path *D*. Path *C* merges with path *D* but opposes path *B*. Thus, the value of two merging movements is more than offset by two opposing movements.

From the foregoing, conclusions may be drawn that, where roadways intersect, the merging movement created by rotary traffic is safest and most efficient and that these advantages gradually reduce as the radius of the rotary movement decreases. If a rotary movement cannot be established so that all movements merge, then a merging intersection or a right-angle intersection (which a rotary movement approaches as a limit) is to be preferred and designs that create opposing movements are best avoided.

Intersection Efficiency.—When continuity of flow (which occurs when intersecting roadways enter a traffic circle so that vehicles merge in a rotary movement) cannot be arranged, certain inefficiencies, which result from interrupted or shuttling movements, must be kept in mind.

The maximum flow capacity of an intersection of two roadways at grade cannot exceed the capacity of the roadway with the greatest number of feeder lanes. Since the number of feeder lanes of important intersecting roadways is usually equal, the maximum possible capacity of a normally important intersection cannot exceed half the capacity of all approach or feeder lanes—that is, of the same intersection with roadways separated (see Fig. 8)—since, if one road keeps full, the traffic on the other has no chance to cross. This 50% maximum efficiency can then be attained only when traffic is flowing continu-

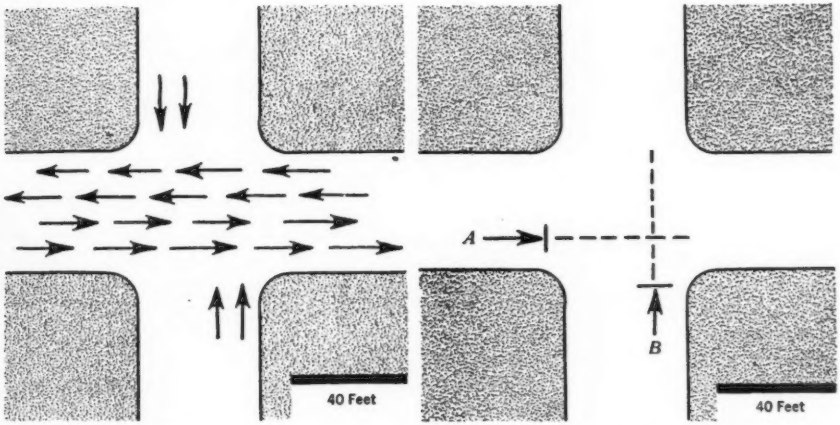

FIG. 8

FIG. 9

ously along one roadway, without interruption. The following discussion of flow efficiency at a normal intersection, where the direction of traffic flow is alternated constantly, is then in terms of some portion of this maximum possible, 50% efficiency, and is based further on the assumption that traffic is feeding in from each direction as fast as the intersection can absorb and clear it.

Assume a simple, right-angle intersection (Fig. 9) equipped, for purposes of this example, with stop-and-go signals. When the red signal stops traffic *A*, a 3-sec or longer intermediate clearance interval is allowed normally for that traffic to clear the intersection, after which traffic *B* gets the green "go" signal. This 3 sec is lost because the main movement, during this time, is at a standstill in both directions. If the cycle of signal operation is 60 sec, this clearance interval time represents 5% dead loss per reversal. Since there are two reversals of traffic movement per cycle, a total loss of twice 5%, or 10%, occurs, per signal cycle, at the usual simple right-angle intersection, in clearing the intersection so that alternate crossing movements may be released.

When traffic *B* gets the green signal, cars in line do not spring into instant motion. Unless he "jumps" the amber light, the first driver will normally lag

in starting from a half to a full second, after the green signal appears. The second car will lag, in starting, behind the first and so on. Thus, under heavy load conditions, a favorable green signal will complete its interval and change again to red by the time a line of cars has "accordianed" out and the end cars are started. Therefore, if a flow of vehicles is stopped and then restarted a considerable period elapses before efficient lane flow is again attained. Principal reasons are driver perception and reaction lag in starting, acceleration lag, a confusion or eddy current lag, and, at intersections, delays arising from turning movements abnormally impeded by the "bunching" of cars.

It will be obvious from the foregoing that deceleration and clearance losses in stopping a flow of vehicles in one direction, and losses in getting started by the crossing movement, introduce a very definite inefficiency every time traffic movements are reversed. These losses would not occur (as an example) if the crossing roadways were separated.

Consider now any intersection which is not at a right angle, as Fig. 10. Traffic *C*, with much farther to go to clear the intersection, takes 5 sec to clear. This adds to 10 sec lost in a two-interval cycle or $16\frac{2}{3}\%$ loss, if a 60-sec cycle is used. Traffic *D* is as inefficient in starting as before and now has farther to

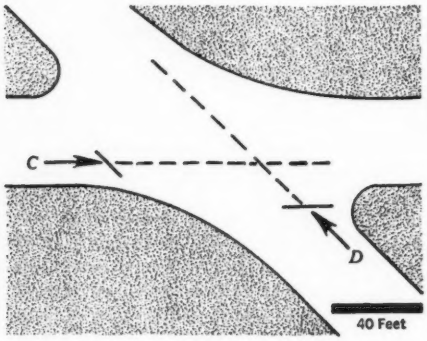

FIG. 10

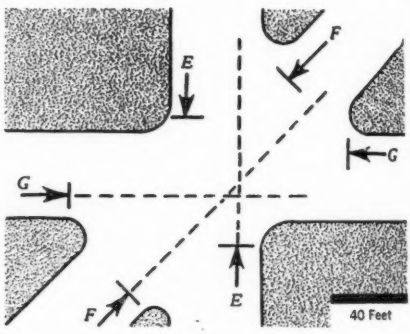

FIG. 11

go to refill the intersection, thus putting it back into full use. Therefore, intersections at other than right angles are even less efficient, when considered in terms of alternating movements, than intersections where roadways do cross at right angles.

At complicated intersections, as in Fig. 11, where three-interval cycles are used, these losses are often greater. Traffic *G* requires at least 5 sec to clear the intersection when the green light disappears. Traffic *E* is subject to the same losses, as shown for Fig. 10, in restarting and in again attaining reasonable flow efficiency. These losses occur in each reversal. Thus, in a three-interval cycle at this intersection of, say, 75 sec total length, 15 sec or 20% may easily be lost per cycle in stopping and clearing each movement to make way for one of the other crossing movements. If the losses, occurring when a line of vehicles restarts, are added to the stopping and clearing losses it will be appreciated that a two-interval signal cycle is to be preferred to a three-interval cycle.

An objective of channelization is to create conditions favorable to the most efficient traffic flow at each location by simplifying movements of intersecting vehicles, by reducing the range of alternation or amplitude of shuttling, by channelizing to reduce to a minimum, the number of reversals per cycle where signals are used, and, if a continuous merging movement cannot be effected, by channelizing traffic flow to insure that vehicles intersect at or near a right angle. Thus, in the case of the multiple intersection shown by Fig. 11, channelization can be used often to establish merging movements or to reduce the range of alternation or shuttling quite substantially and, if signals are necessary at this intersection, to reduce the usual three movements to two per signal cycle.

Left Turn.—Theoretically, all cars turning left should circle the center of an intersection. Practically, another type of left turn is usually more desirable. As shown in Fig. 12, cars turn left around the center at such short radius that even those with short wheel bases find the turn difficult, whereas a truck, or a car with a long wheel base, often finds it impossible to make the turn. It will be

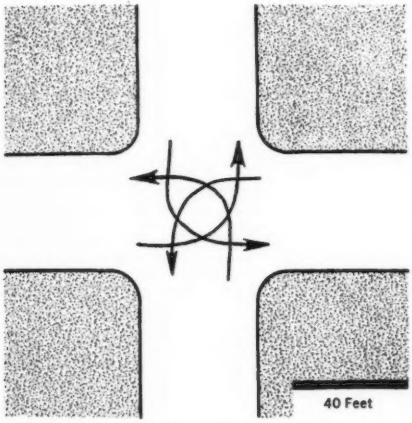

FIG. 12

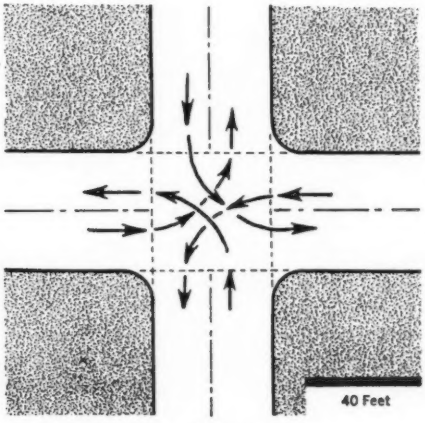

FIG. 13

noted that cars turning left, from each direction, describe paths that intersect at eight points. With this type of turn, intersection "jams" or "lock ups" occur frequently.

Another, and much more natural left turn, is one in which the north-bound car, Fig. 13, turning left (west), enters the intersection from a position to the right of the street center line and leaves to the right of the center line of the street on the left. All four cars, each turning left, describe paths which intersect at only four points. This "inside left turn" is efficient, faster, safe, and natural.

Channelization creates conditions at intersections which permit the "inside left turn" but make its abuse difficult. Some of these abuses are shown in the following examples.

Approach from Behind and Eddy Currents.—A traffic accident is not a premeditated act but occurs when something unexpected and unlooked for happens. One course of least resistance that results in unexpected traffic movements, disorder, and accidents at intersections is the short-cut, left turn.

Obeying a natural impulse, a driver follows the most convenient, straight-line or long-radius course in an unrestricted intersection or traffic area. If (as is usually the case) the course followed is a wrong one, he goes too fast, operates in and across wrong lanes and comes unexpectedly upon other cars from head-on, sum-of-the-speed angles.

Driver *A*, Fig. 14, cuts a short corner. Taking a long-radius course, his speed is high. He gets into two wrong channels. Like a man walking diagonally across a street intersection, he must now look in three directions at once to watch out for the possibility of cars *B*, *C*, and *D*. Driver *B*'s attention at the moment is properly devoted to driver *C*. Driver *C* gives his attention first in the direction of driver *D* and then in the direction of driver *B*. None of them is looking for, or expecting, the movement of driver *A*. While car *A* merges with car *D*, it intersects the courses of cars *B* and *C* at opposing, sum-of-the-speeds angles. Pedestrians *W*, *X*, *Y*, and *Z* are watching cars *B* and *C*. They do not expect, or look for, car *A*. In each case driver *A* comes upon other drivers and pedestrians unexpectedly and from behind their respective ranges of vision. Drivers *B*, *C*, and *D* and pedestrians *W*, *X*, *Y*, and *Z* abandon natural or instinctive driving and walking habits and do unexpected things that now make them dangerous to others and to themselves and which are characteristic of the disorderly and hazardous, traffic eddy current. Such eddy currents are progressive and tend to increase from minor to major complication.

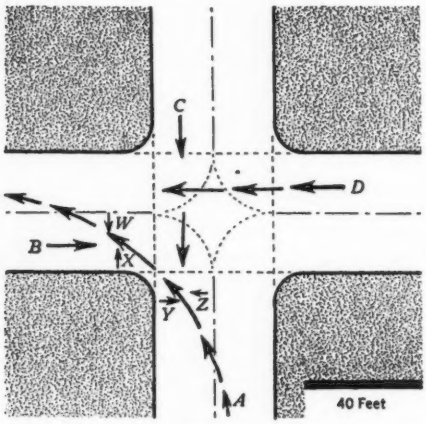

FIG. 14

Traffic eddy currents are illustrated by the sequence of events leading to an actual accident shown in Fig. 15. Driver *A*, making a long-radius, short-cut turn at speed, passed driver *E*. Because visibility was obscured by a building, driver *A* did not, at first, see driver *B*. With right of way over driver *C* and looking in the direction of driver *C*, driver *B* started across the intersection. Driver *A*, to avoid driver *B*, deflected right forcing driver *C* to the left, and driver

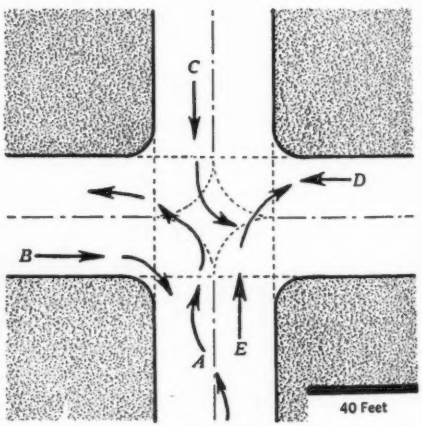

FIG. 15

E, dodging driver *C*, crashed into car *D*. This eddy current movement in which one car, cutting short and from behind, forced other cars progressively into unexpected opposition at sum-of-the-speed angles, is a common occurrence which may be observed in some degree at any ordinary intersection, if the movement tracks of cars are traced, during a period of light traffic, on an intersection diagram.

In round numbers, 75% of traffic accidents in cities and 25% on highways occur at intersections. Of course, there are many more intersections per mile of street than per mile of road so this percentage difference does not indicate that a highway intersection is less dangerous than one in town. Irregular movements at intersections, arising principally out of high-speed, short cuts and turns, or improper passing, are a major cause of these intersection accidents and of the inefficient traffic flow so often conspicuously apparent. Channelization, properly applied to simple, as well as to complicated, intersections, prevents the irregular vehicle movements that are a cause at once of accidents and of inefficient vehicle flow, both of which result from the same disorderly causes.

Right of Way.—The law that gives right of way to the driver on one's right is a little-recognized source of confusion where vehicles intersect.

Under the law that gives right of way to the driver on one's right (Fig. 16), driver *A* has right of way over driver *B*, driver *B* over driver *C*, driver *C* over driver *D*, and driver *D* over driver *A*. In the exercise of this right, a driver may find himself blocked in the middle of an intersection by another driver who is in turn blocked by still another.

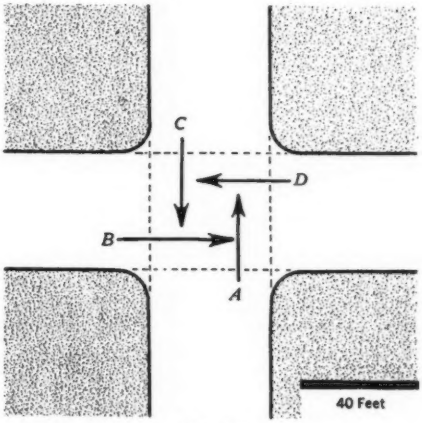

FIG. 16

The law results in other confusions. Driver *A* enters the intersection with, in his judgment, apparent right of way over *B*. If *A* is wise he will give close attention to *B* on his left to avoid collision if *B* assumes he has right of way or fails to obey the law or if, due to speed or other differentials, the question of who has the right of way is uncertain. Concerned with *B* on his left, who is his first and immediate problem on entering the intersection, *A* must often look two ways at once and settle a problem of relative speeds and distances to determine if *D*, who is on his right, is within an approximate right-of-way range and is therefore entitled to it or not.

In this problem (which involves looking two ways at the same time, often from a position within an intersection), swift appraisal of a number of varying factors, and quick decision, confront a driver each time his course intersects those of other vehicles. As a matter of fact, the writer is not familiar with any rule by which a driver can know exactly just when another driver on the right

is entitled to right of way—whether it is when two cars are a block apart, 100 ft apart, or 50 ft. This makes the problem of determination, which is troublesome enough anyway, doubly difficult.

Assume that the law were changed to give right of way to the man on the left, as illustrated by Fig. 17. From a position outside of, rather than within, the intersection, driver *A* gives major attention to *B* who is now not only the one whose path he crosses next but also the driver to whom he must yield right of way if *B* is within or entering the intersection. This points to a suggestion of a definite and easily established reference point, namely: "if *B* is within or entering the intersection" he has the right of way over *A*. When *A* does enter the intersection his right-of-way relationship with *D* is similarly simplified for both drivers; that is, when *A* reaches the curb line he has right of way over *D* if *D* is outside; but he does not if *D* is entering or is inside.

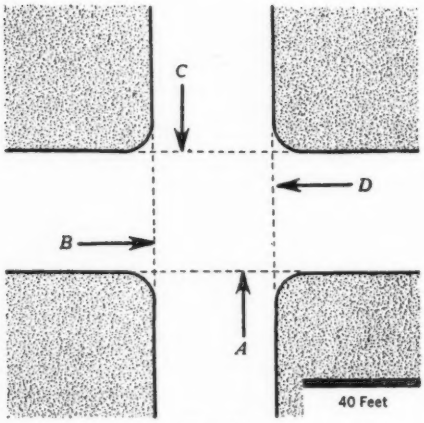

FIG. 17

Thus, if right of way were given to the driver on the left, center of the intersection, lock ups now resulting from the opposite rule would be avoided, and the problem of who has right of way would be simplified greatly. Problems arising from having to decide two things, and to look two ways at once—toward the man on one's left whose course one intersects next and the man on one's right who is entitled to right of way— would be reduced or avoided.

Furthermore, the question of who is to have the right of way next would be determined by drivers from a distance equal to the roadway width apart, before they enter the intersection, and therefore from a position of safety. Drivers would not have to make this decision as they are entering or are actually in the intersection exposed to collision or, in any case, in close range contact with other vehicles, as is the case under the present rule.

When cars travel on the left side of the road and the driver sits in the right side of the car as in England, the rule that gives right of way to a driver on the right is easily obeyed because it is instinctive. In the United States, where one drives to the right and sits in the left side of the car, to yield right of way to a driver on the left is instinctive and natural whereas giving right of way to a driver on the right is unnatural, less efficient, and unsafe.

Channelization cannot correct, but it may often be applied to modify, confusions arising from this reversed regulation at ordinary intersections. It reduces confusion caused by this regulation in large traffic areas. This is particularly true where, in a channelization treatment, merging movements of vehicles can be arranged.

Applications

The foregoing is an approach to necessary understandings and points of view of the engineer who studies a traffic situation with the object of creating a design of roadway that blends driving requirements and driver characteristics into a satisfactory solution. Such an engineer understands that channelizing, properly done, is a form of regulation by which right actions are made easy and wrong ones difficult. He knows that agreeable restraints, to prevent irregular movements of vehicles, establish order that avoids accidents and promotes flow efficiency. A number of actual examples follow. These start with simple problems and progress by steps through more complicated and difficult situations.

Simple Intersections.—Accident hazard is greatest as to likelihood and severity when traffic is light, fast, and unrestrained. Hazard declines as traffic density increases, up to the point of such dense traffic that vehicles scarcely move, in which case hazard approaches zero. In many business districts, less than half the accidents occur during business hours and the remainder (which are also the most serious) occur when the business day is over and traffic load is lightest. If vehicle track patterns in intersections are traced while the traffic load is heavy, the movement of cars will be observed to be quite orderly and, for the most part, proper. As traffic density declines, speeds increase, and patterns become increasingly irregular, as indicated by Fig. 18.

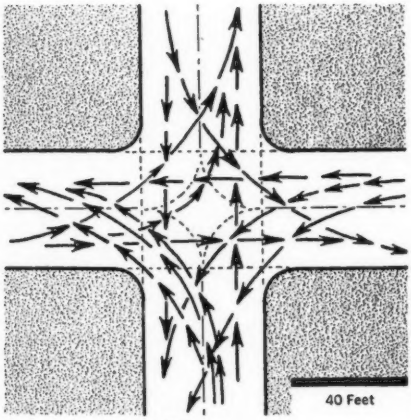

Fig. 18

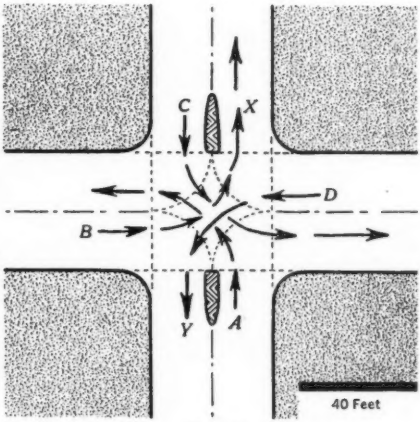

Fig. 19

Left turns particularly change from orderly movements to high speed, short cuts.

One function of channelization consists in the treatment of ordinary intersections to correct these disorders. Thus, the most convenient course for car *A* in clearing the lower island, Fig. 19, is to drive straight ahead and to begin a left turn at or beyond the end of the island. Starting a turn from this position and making it at maximum comfortable speed (which drivers may be counted on to do), car *A* will enter the street to the left correctly, even if it is a wide one—that is, to the right of its center line.

Similarly, driver *D*, turning left, will clear the lower island most conveniently by continuing into the intersection to the right of the center line of the street he is leaving, thus making a natural turn and dropping straight in past the island. The upper island similarly regulates the movements of vehicles *B* and *C*. Thus two islands placed in opposite throats prevent irregular turning movements in the intersection.

To state this in another manner: Drivers *A*, *B*, *C*, and *D* find correct movements easy and natural and wrong ones not only awkward but actually difficult and inconvenient. Pedestrians are safeguarded by islands of refuge and by the fact that traffic movements are orderly and predictable.

If stop-and-go signals are used at this intersection they may often be installed, particularly in cities and towns, on pedestals on these islands where, in addition to their normal functions, they signalize the islands; and, from this position, they usually have best visibility to drivers and to pedestrians. Such treatment has the value, also, that the signal need be operated "stop" and "go" only during rush hours and the channelizing treatment meets requirements, at other times, with signals flashing amber, thus permitting continuous, safe movement of traffic.

Wide Throat Intersections.—Given the opportunity to roam over an enlarged area, drivers accept the invitation and invariably proceed along paths which are the shortest or fastest courses between any two points, as shown in Fig. 20. It will be noted that vehicles intersect at steep, head-on angles, approaching

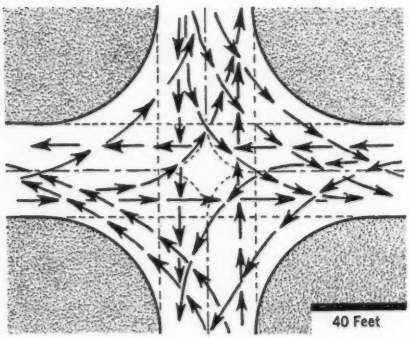

FIG. 20

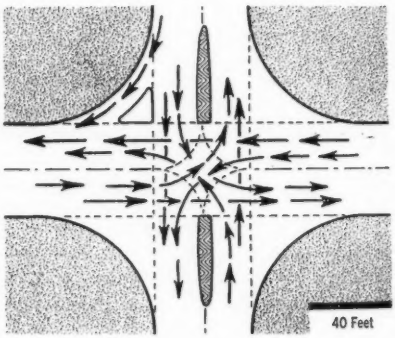

FIG. 21

each other at nearly the sum of their speeds. The short-cut, vehicle paths and speeds "too great for conditions" on left turns are not supposition. If the opportunity is there, it will be taken advantage of, except during the periods of heavy load.

The treatment used in the correction must be carried from the curb line back somewhat behind the point where the roadway is of normal width (see Fig. 21); otherwise the invitation to cut short remains. The short cut must be blocked.

Enlarged Throat, Dead Ends.—The short-cut, long-radius, high-speed movements indicated in Fig. 22, produce head-on, sum-of-the-speed, vehicle inter-

sections. Pedestrians, particularly, have a difficult time in these situations because it is often not clear, at any point, from which direction the next car will come. Drivers, also, find themselves in the same predicament.

Two corrective treatments are indicated in Figs. 23 and 24. It will be noted that both treatments block wrong courses and make right ones easy and inviting.

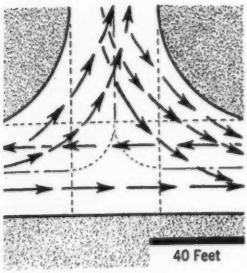

FIG. 22

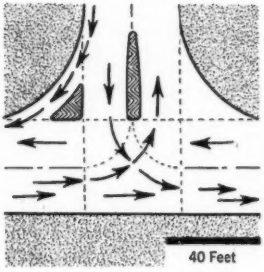

FIG. 23

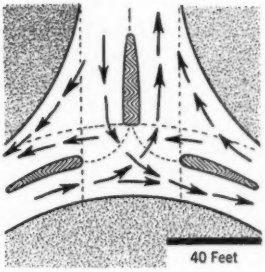

FIG. 24

Note, too, that, in all cases, intersection of vehicle paths is about at right angles and that turning movements are easy and normal. Pedestrian refuge islands on the sides as indicated on Fig. 23 may be desirable sometimes.

Offset Intersections.—The type of intersection shown in Fig. 25 usually develops the compound crisscross of movements indicated. Examination of

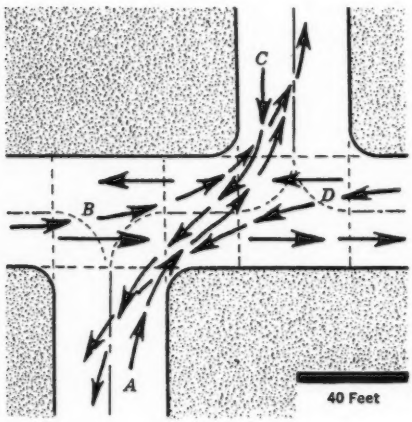

FIG. 25

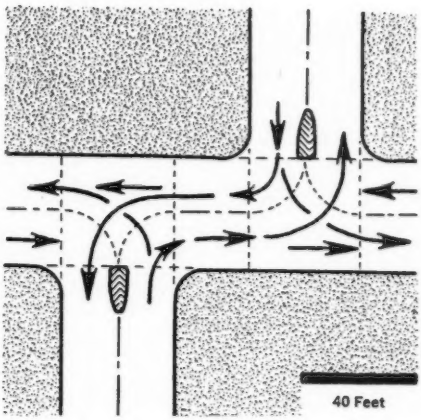

FIG. 26

curb faces at the lower right and upper left hand corners often discloses abrasion and tire marks from traffic *A* and *B* which strike at the upper left and traffic *C* and *D* which strike at the lower right. Wheel-mark patterns on the pavement usually confirm how completely offside drivers do get when the intersection layout invites such movements. The treatment shown in Fig. 26 corrects faulty movements and provides pedestrian safety in the two offset street throats as well.

Angled Roadway Intersections.—The angled street type of intersection invites a long, speedy short cut in which cars *A* and *B*, Fig. 27, approach head on. Because of the distance between curbs, the pedestrian hazard is greater than

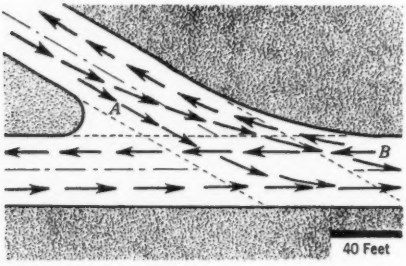

FIG. 27

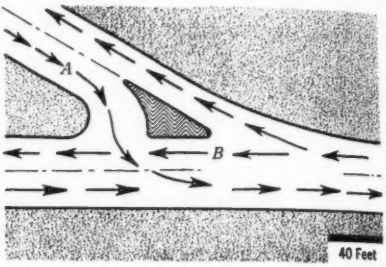

FIG. 28

normal. If no left turns are made into the angled roadway the treatment indicated in Fig. 28 is effective. Where left turns are a factor, as in Fig. 29, the island may be reduced to permit a free, easy-radius, left turn. In cases of heavy pedestrian traffic, the right-hand island, as shown in Fig. 29, may be desirable. In any of these treatments, pedestrians, as well as drivers, have only one direction to look at a time. Traffic *A*, the conflicting element, without

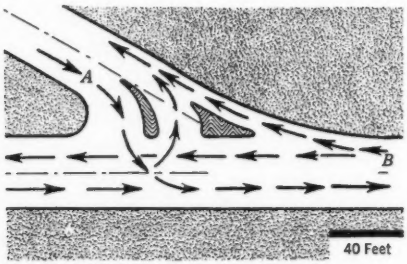

FIG. 29

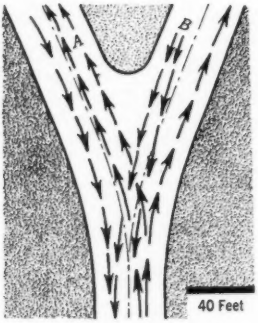

FIG. 30

being unreasonably inconvenienced, is forced into a right-angle intersection, at a moderate speed, with traffic *B*, and is no longer a menace to pedestrians, to other drivers, and to itself.

Y-*Intersections.*—Although the construction of Y-intersections should be avoided, many exist to plague the driver and the traffic official (see Fig. 30). They invite the type of head-on intersections of vehicles at steep, sum-of-the-speed angles that are the cause of continuous disorder. Such conditions render judgment difficult and change instinctive and natural driving to the uncertain, unpredictable movements of a driver who proceeds on the basis of separate observations and of separately thought out actions. Such drivers, whose timing now goes askew and who do erratic things, are often mistakenly charged with carelessness.

Treatments shown in Figs. 31, 32, and 33 block wrong courses. They cause traffic *A* and *B* to intersect at a right angle and at an observable, predictable point. Note that the axis of the treatment may be swung to accommodate

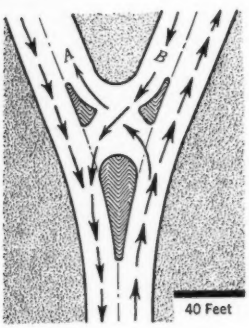

FIG. 31

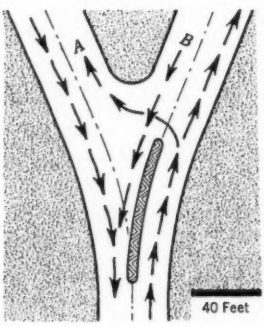

FIG. 32

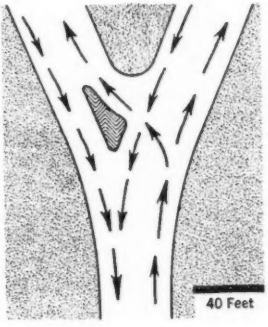

FIG. 33

an equally distributed load, or to give greater convenience to a heavy traffic load in one direction. Note, too, that islands used extend back far enough to prevent cutting short.

X-*Intersections.*—The **X**-intersection is a double **Y** with two extra head-on movements added so that the problem is compounded (see Fig. 34). Drivers

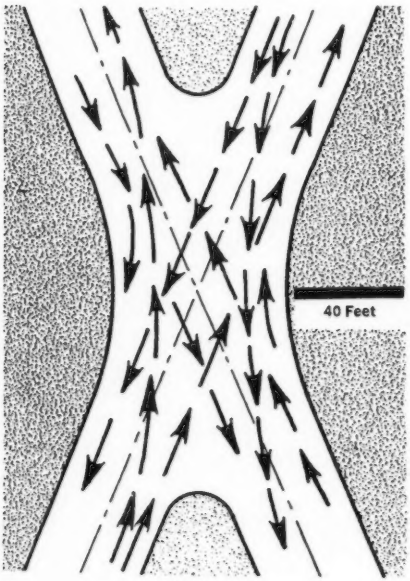

FIG. 34

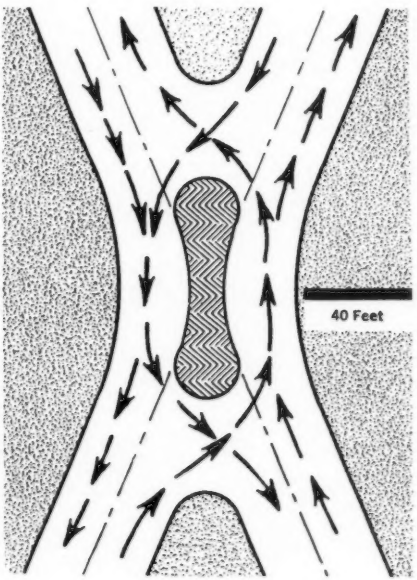

FIG. 35

often find instinctive judgments incorrect because the actions of other drivers are unnatural and unpredictable. An eddy current involving one or two cars

suddenly spreads to a number, for quite a distance back in two or more directions; and chaos, which always threatens, often follows.

The axis of the correction is usually arranged to favor heaviest load, as indicated in Figs. 35, 36, and 37. Definite reference and intersection points

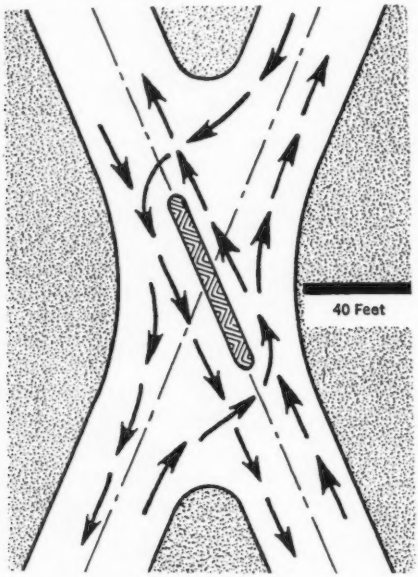

FIG. 36

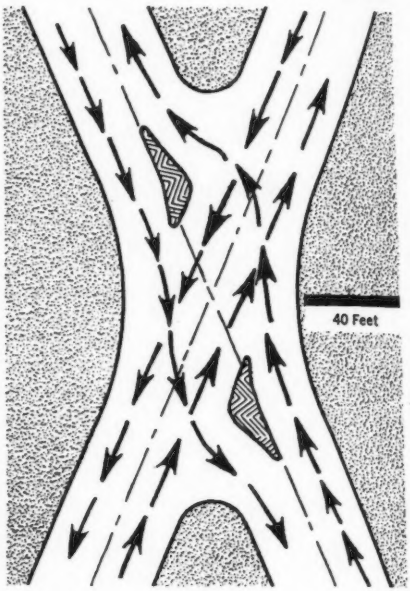

FIG. 37

are established and orderly movements give instinctive driving abilities more chance.

Treatment to Favor Traffic Load.—A good example of swinging the axis of a traffic island treatment to favor the heavier load is to be found in the "Miracle Mile" near Phoenix, Ariz. (see Fig. 38). In this case, the main loading *A* and

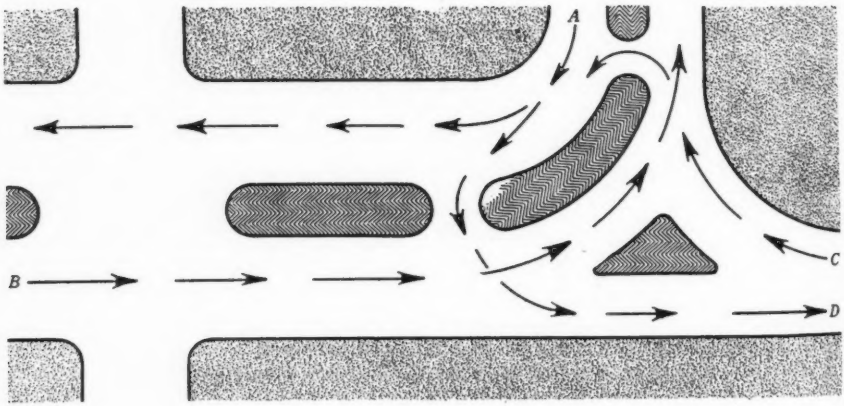

FIG. 38

B is convenienced, whereas *C* and *D* are forced into slower paths which merge in the case of *C*, *B*, and *A* and, in the case of *B* and *D*, intersect on the merging side of a right angle.

Parkway Intersections.—The center parkway type of intersection, shown in Fig. 39, invariably breeds much trouble for both pedestrians and drivers.

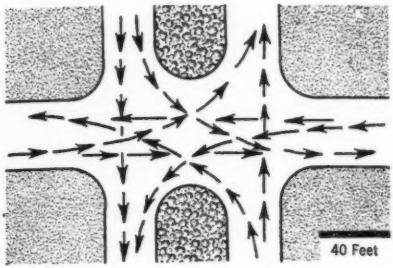

FIG. 39

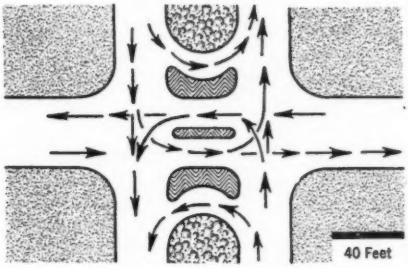

FIG. 40

Vehicle courses, usually shown quite clearly by wheel tracks on the pavement explain why.

Special difficulties introduced by the center parkway are easily corrected by the installation of the center island, Fig. 40, which, if properly located, prevents irregular movements and invariably eliminates the problem.

Grand Circus Parkway, Detroit, Mich., is a major city street with a wide parkway down the center. There is a heavy U-turning movement of traffic to hotels, offices, and shopping districts. "Turn through" openings are provided in the parkway as shown in Fig. 40. These openings may be near the intersections or closer to the center of the block, as conditions dictate. In either case, they permit a car to execute a U-turn without crossing pedestrian lanes or getting into interference with traffic in the intersections. These "turn through" openings also provide a means by which left turns may be eliminated at intersections of parkway streets or divided highways. In such a case a driver, wishing to turn left, proceeds beyond his intended roadway to a "turn through" from which he merges with opposite traffic, back tracks to his road or street, and turns right. Cross traffic intending to turn left is accommodated similarily.

Multiple Intersections.—Multiple intersections (Fig. 41) frequently present very complicated problems of safety and flow efficiency for both vehicles and pedestrians. If there must be such intersections, there are often quite simple channelization treatments which remove many of the difficulties and simplify the problem so that, if further correction is needed, simple signal-control methods meet any remaining requirements quite satisfactorily.

The half circle or built-up sidewalk treatment, as shown in Fig. 42, may be used, particularly where main flow of traffic is to be favored. This solves most, if not all, pedestrian and vehicular difficulties.

Quite often an original design, or a redesign of an existing situation, will permit the installation of a complete circle with deflection islands, as in Fig. 43. In such cases a very satisfactory solution is the result because resulting order prevents accidents and insures flow efficiency.

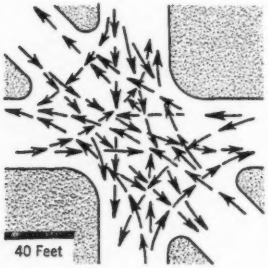

FIG. 41

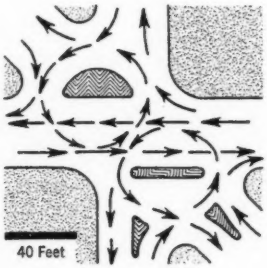

FIG. 42

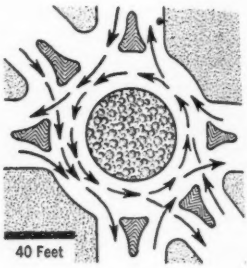

FIG. 43

Large Traffic Area.—Channelization of traffic areas is well illustrated by the problem shown in Figs. 44 and 45, which is an outstanding example of results to be accomplished by simplifying traffic flow and applying other principles referred to in this paper.

Prior to March, 1921, in the Lincoln Monument area of Fairmount Park, Philadelphia, Pa., three officers and a sergeant were used to handle traffic during rush hours. Each driver suited his own convenience as to routing. When the

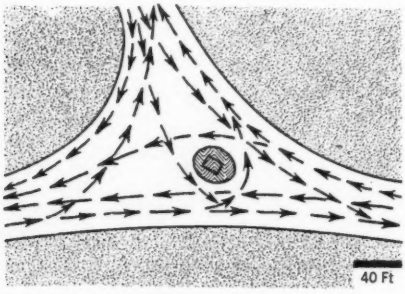

FIG. 44

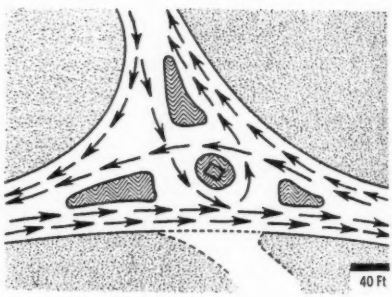

FIG. 45

flow of cars was heavy, congestion and lock ups occurred; at other times speed became a factor. There were nine major points of intersection at which vehicles approached each other from head-on, sum-of-the-speeds angles. Since there were no definite channels for traffic, a driver could not be sure what others would do. Resulting disorder caused accidents, tangles, and confusion; and, during rush hours, it was the cause of serious congestion with long lines of blocked cars, particularly in the direction of the heavy flow.

The channelizing treatment, shown in Fig. 45, simplified traffic flow. It confined vehicles to definite channels, obstructed wrong movements; proper paths were made convenient, many intersection points were reduced to three

in which vehicles intersected at right angles, pedestrians were provided with safety zones, unexpected high-speed approaches from behind were eliminated, and drivers and pedestrians had the confidence of knowing what to expect and where to look. Note that the island on the left shades traffic turning left around the monument into East River Drive. This offset is needed at this point and proves remarkably effective.

This simple treatment eliminated the four traffic officers and, since 1921, has handled, with no difficulty, many times the volume of traffic which moves continuously from and to all directions. Since early in 1937 a park guard was stationed, during the morning rush, at the right-angled corner of the left-hand island, and during the evening rush at the right-angled corner of the upper island. His function is not to stop traffic but to prevent momentary blockades at these points.

In the summer of 1938 another outlet to this intersection, which is used only slightly, was added as indicated by the dotted line.

Divided Highways.—The divided highway is a channelizing device by which traffic flowing in opposite directions is separated by a physical barrier such as a center island or curb. The principle of separation of opposite flows of traffic is capable of enlargement to accomplish other functions including that of rotary movement of intersecting traffic. Since this type of treatment has been covered most ably by the late Fritz Malcher,[4] the writer's observations are limited to certain directly interesting and important aspects.

Several applications of the divided highway are encountered in practice. One method, illustrated in Fig. 46, provides openings in the center island for

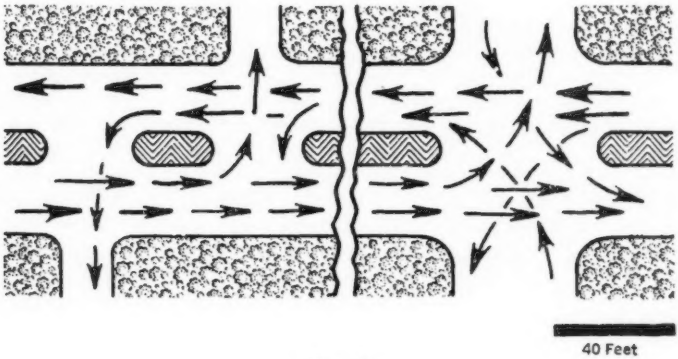

FIG. 46

crossing into or out of every driveway and street. In this case, much of the advantage of the divided highway is lost because through drivers face the constant double threat that a car in one high-speed lane may slow down and pass through one of the many dividing spaces into or across the other high-speed lane in entering or leaving a local entrance. On a divided highway with many center openings, a moment's inattention may be serious because the entering

[4] "The Steady-Flow Traffic System," by Fritz Malcher, Harvard City Planning Studies, Vol. IX, 1935.

and leaving movements referred to, into and out of the high-speed lanes, are often wholly unexpected, confusing, and dangerous, particularly after dark.

Another method (shown in Fig. 47) considers that the convenience and safety of the heavy flow of traffic on the main highway are more important than

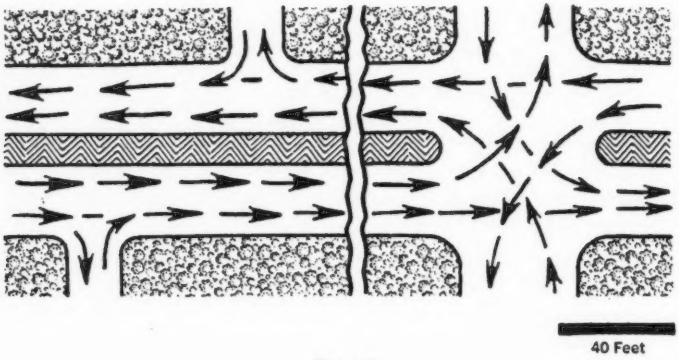

FIG. 47

the convenience of a few vehicles from the side, if they are not too greatly imposed upon, and provides openings through the center or barrier section only at intersections. This reduces the points at which main flow traffic need concern itself about crossing vehicles and concentrates cross flow so that "stop" and "go" signals, or other control measures, may be provided and justified.

A third method (see Fig. 48) provides openings through the center barrier at intervals often 1,000 to 1,500 ft or more apart. Such openings should be

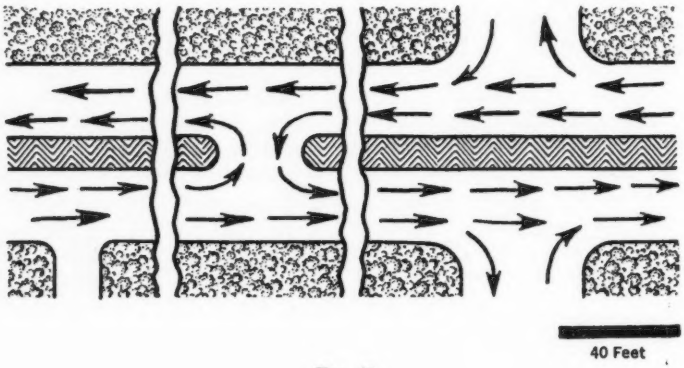

FIG. 48

midway between, or at least some distance from, intersecting streets or roads. This establishes a form of rotary traffic which, if handled properly, results in merging movements of cross and through traffic. The greater safety, promptness, and certainty of safe departure from, entrance into, and crossing major highways, with this treatment, may be expected to find support from both local and through motorists, particularly on some main four-lane highways that

carry such heavy loads that sometimes it is almost impossible to cross them without the aid of an officer or a "stop" and "go" signal.

One important factor in this treatment consists in a dividing island that is at least wide enough, or has barrier openings that are long enough, or has a combination of length and width, so that vehicles can cut into the center openings, out of a high-speed inside lane, quickly, with least disturbance. Vehicles can then wait, without projecting into a lane of traffic on either side, until an opportunity presents itself to merge into the high-speed lane on the other side.

This treatment may be extended a step further. In many cases, the problem of intersecting and departing vehicles on major, multiple-lane, highways could be met by providing sufficient right of way so that a very safe rotary movement of crossing and turning vehicles would be established. To be satisfactory, accelerating and decelerating lanes should be provided for vehicles merging with, or leaving, the main flow. Such an arrangement is indicated in Fig. 49. This method permits continuous safe movement of intersecting and main-flow vehicles with least interference or danger. The acceleration and deceleration strips should be long enough (350 ft or more, depending on normal traffic speeds); they should be of a color that is in contrast to the main roadway, and should preferably be unattractive to the high-speed, main movement and yet be inviting to traffic merging into, or departing from, the main highway.

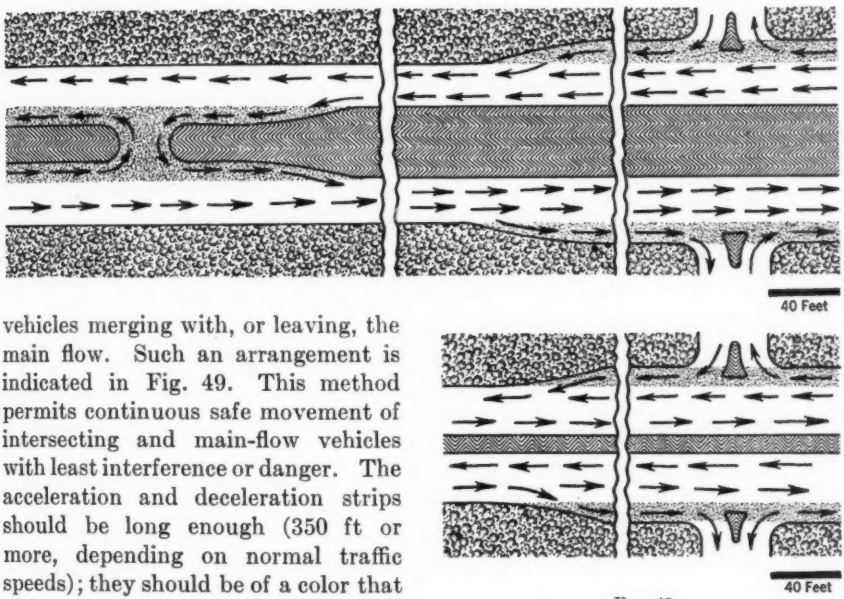

FIG. 49

Where, on main highways, openings through the center islands can be spaced sufficiently far apart, the center island may be tapered down to slight width, in center sections, as shown, thus reducing land use and overpass or bridge widths. The designers of the Merritt Parkway in Connecticut used this method of providing ample turning radius through island openings, of saving valuable land space between, and of reducing overpass and underpass widths.

Grade Separation.—At the intersection of major roadways, sometimes channelization may be used to postpone the need of providing costly grade

separations (see Fig. 50). Since such need is usually anticipated well in advance and long before the intersection becomes a major problem, sufficient land may be acquired at such locations to permit the installation of an adequate traffic circle. When a separation finally does become necessary, the need for it will have been postponed, the land, originally secured at a lower cost, will have been utilized for rotary traffic in the interval, and a sufficient land area will have been provided for the separation and approaches.

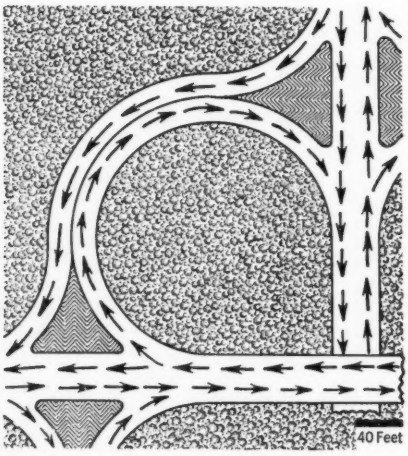

FIG. 50

The sketch, which is used in this instance as a symbol (Fig. 50), shows a quadrant section of a typical grade-separation of the clover-leaf type. Many variations of this treatment are used. All depend for their success on channelization principles. Channelization, therefore, is not only necessary to the success of these advanced practices in highway design but is an element in postponing or eliminating the need for such expensive treatments and in reducing their final cost.

PEDESTRIANS

The problem of the pedestrian has not been placed last because it is considered unimportant, but for the reason that, if complicated vehicle movements are made orderly, the question of what to do with, or about, the pedestrian is solved—or at least simplified. The pedestrian was not born confused. Rather he is a creature of his environment, the victim of the high velocities and unexpected attacks to which modern traffic exposes him. These are the cause of his uncertain movements and hesitations.

Channelization removes the element of surprise from the pedestrian problem. Proper channelizing insures the pedestrian that he need look only in one direction at a time and that he will not be "ambushed" by vehicles that attack suddenly from behind his range of vision. Also he is provided with an island or refuge of ample width at intervals of one or two traffic lanes upon which he can orient himself, safely, before he crosses the next channel in which, also, vehicles will move in one direction only.

When the confusions, fears, surprises, "ambushes," and alarms introduced by vehicles moving swiftly from expected and unexpected directions are eliminated, the pedestrian is master of his own destiny to a surprising extent.

After establishing orderly traffic movements and pedestrian refuges by channelizing, if foot passengers are still in need of a special officer, signal, or other aid at any point, such aid will usually be necessary only to a limited degree and during certain hours rather than for long, or all-day, periods.

Conclusions

Beginning with the principles applied in handling bicycle traffic, horse-drawn vehicles, and early automobiles, there has been a slow but continuous evolution of channelization until today these principles are being applied in more advanced forms on major roadways, in the reorganization of many existing intersections and traffic areas, and in the solution of traffic difficulties at intersections where such treatments eliminate the need for redesign and reconstruction.

Undoubtedly, designers of major highways in the future will make use of the continuous flow principle in which a center barrier strip divides the highway, and openings occur at intervals, but not opposite or closely adjacent to intersections. Thus a truly rotary, merging movement of crossing traffic is provided.

At important highway intersections it seems probable too that, if grade separation is not advisable immediately, sufficient ground will be acquired (as is extensively done in New Jersey where a large number are in use) to establish a traffic circle large enough to provide a continuous, smoothly merging flow of vehicles. Later, if traffic attains volume and speed enough to require a grade separation, that day will be deferred by the efficiency of the rotary movement. The circle will provide the land, initially secured at lower cost, in the meantime serving a valuable purpose, upon which to place a grade separation.

By permitting continuous, orderly flow at reasonable speeds, channelization helps drivers to their destinations more quickly and safely than the dash-slow-down-dash-stop-and-dash procedure otherwise necessary.

A more general application of these principles will assure fluid movement of motor vehicles and will reduce, greatly, the confusions, and eddy currents that are the cause of accidents.

Plans for, and the administration of, motor transport of the future must include greater attention to driver characteristics and to his problems and needs—that is, to the problems of traffic from the pavement surface up. To accomplish these purposes, a greatly expanded knowledge and application of channelization will be necessary.

Acknowledgments

In addition to names cited directly in the text, valuable suggestions, ideas, and criticisms were received from other engineers, traffic officials, and laymen almost too numerous to mention. At various times, officials of such state highway bodies as Arizona, Maryland, Minnesota, New York, Ohio, and Washington, and traffic experts of the cities of Philadelphia, Pa., Kansas City, Kans., and Chicago, Ill., and of the American Automobile Association, the National Paving Brick Association, and the American Transit Association, have contributed to the work.

These acknowledgments would be incomplete without mention of the writer's immediate associates of many years in the American Gas Accumulator Company and Signal Service Corporation, who helped to establish, refine, and define the principles of channelization described in this paper and to evolve the language by which its requirements have been expressed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

WATER SUPPLY ON UPPER SALT RIVER, ARIZONA

By John Girand,[1] Assoc. M. Am. Soc. C. E.

Synopsis

Presented in this paper are the data and methods used to determine economical storage requirements and power outputs of a proposed hydroelectric plant on the Upper Salt River, Arizona. The gaging station at the project site had only been in operation for a few years, necessitating a correlation of data which covered a longer period of time, including other stream-gage records, rainfall records, and tree-ring measures.

After the hydrograph of the river flows had been constructed, the storage requirements and power output of the plant were estimated by a new method involving the construction of a probable future hydrograph, based on laws of probable occurrence and well-defined trends of long-period variations in river flows. Due to the greater value of water in the semi-arid regions, particularly in periods of low flow, special treatment has been given to drought periods involving methods of probable sequence of drought years.

A copy of the paper, with two appendices containing detailed observations of runoff and rainfall, has been placed on file for reference at Engineering Societies Library in New York, N. Y.[2]

Introduction

The Salt River rises at an elevation of 10,000 ft above sea level in the White Mountains near the eastern boundary of the State of Arizona. It flows generally westward through heavily timbered plateau country with an average elevation of 6,000 ft until it reaches the Mogollon Rim, where it drops rapidly in a few miles to an elevation of 3,000 ft. Then it flows on a flatter grade through rolling and sparsely vegetated country into the Roosevelt reservoir near the central part of the state. The water impounded in the Roosevelt

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by April 15, 1940.

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[2] 33 West 39th Street, New York, N. Y.

reservoir is used for irrigation in the Salt River Valley (population, 100,000) and for the generation of power used in Phoenix and vicinity.

The relative isolation of the Salt River Valley results in high costs for fuels required for prime movers, and has lead to an intensive development of the hydroelectric resources of the state. Five hydroelectric plants are now (1939) operating on the Lower Salt River below the Roosevelt Dam with an installed generating capacity of 79,000 kw.

In spite of the intensive development of water resources, rapid increase in agricultural and industrial consumption of power has caused a shortage in the Salt River Valley. Heretofore, investigation of power possibilities on the Salt River above the Roosevelt Dam was not considered seriously, as good sites were available below Roosevelt Dam which would have the beneficial effect of regulation by storage above the power dams. At present, with the river completely regulated below the dam, as a matter of necessity the Upper Salt River was investigated.

In the canyon section, the Upper Salt River has a steep gradient and in places is extremely tortuous. In one place, Mule Hoof Bend, the river makes a 3-mile loop and returns within 400 ft of itself, with a 75-ft difference in elevation in water surfaces. At another place, Walnut Canyon, a conduit one mile long will cut off 3 miles of river and give a potential head of 300 ft. In this canyon section there are many excellent dam sites and power house locations; a main highway, constructed in 1937, gives easy access, and the project was seen at once to be capable of producing power very economically.

Due to the fact that there are a number of units which might be built economically, each operating under a different head, the water supply study was made on the basis of a unit head of 100 ft from which the output of any desired plant could be determined by multiplication by the proper head factor.

The Hydrograph

A gaging station has been maintained at the project site only since 1924. Below the project there are other gages which have been in operation intermittently since 1888. The locations of the various gaging stations are shown in Fig. 1.

Estimated values were determined from the direct relation between Chrysotile gage and other individual gages on the river, by plotting the simultaneous monthly values of the Chrysotile gage and the other gage under consideration. Logarithmic scales were used to keep the percentage of accuracy constant for all values of flows, as well as to allow determination of an equation of correlation in exponential form. From these graphs, the record of runoff at the Chrysotile gage was estimated for those years previous to 1924, which were covered by the Roosevelt and McDowell gages.

During 1889 and 1890 the only records obtained on the river were taken from observations at Arizona Dam, below the junction of the Salt and Verde rivers. The Verde River drains country with different topographic and climatic conditions; and the problem, first, of separating the flows at Arizona Dam into the Salt and Verde river components and, second, of reducing the Salt River com-

ponent to a Chrysotile equivalent, introduced so many unknowns that with the short period of record available a satisfactory relation was not found.

Although it may be possible to determine a relation without too great an error, the accuracy of the data is questionable. The original observers' records

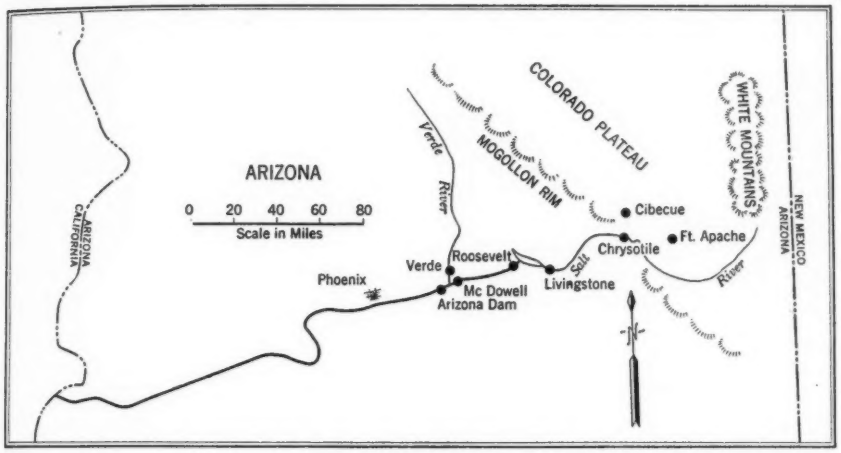

FIG. 1.—LOCATION OF GAGING STATIONS

show the discharges in cubic feet per second filled out daily with great care and decimal point exactitude at a time when Apache Indians were active in this area, leading to the conclusion that the flows were inserted by a not too conscientious observer. Furthermore, it is stated that the measurements were made in the canal headgates and that flood flows were only estimated and not measured. The data at hand allowed the construction of a hydrograph of estimated flows at the project site from January, 1891, to September, 1936 (see Fig. 2).

DATA DERIVED FROM THE HYDROGRAPH

In the foregoing, the data available were stated to be correlated to obtain a hydrograph of the estimated flow of the river at the project site during the forty-five years since 1891. With the hydrograph in Fig. 2 there were two general methods of analysis whereby an estimate of the future power output of the project might be obtained, either of which methods would not include an important factor in storage reservoir operation.

The mass graph, a summation of flows as they occurred in the past, treats the data in strictly chronological order. Probability curves and the probability derivative, the duration curve, treat the same data without respect to chronology. With either method, the important fact which is lacking is the sequence of flows to be expected.

Although it is well known that the mass graph treats the data in a sequence of flows as they occurred in the past, there is no evidence to indicate that the same sequence will be repeated in the future; even a superficial examination of

the hydrograph will show that no sequence of flows covering a 2-year or a 3-year period has ever been repeated. Thus, an analysis based on a mass graph of past runoff, although admittedly giving a solution that would be most economical for a sequence of flows as they occurred in the past, would not necessarily give a

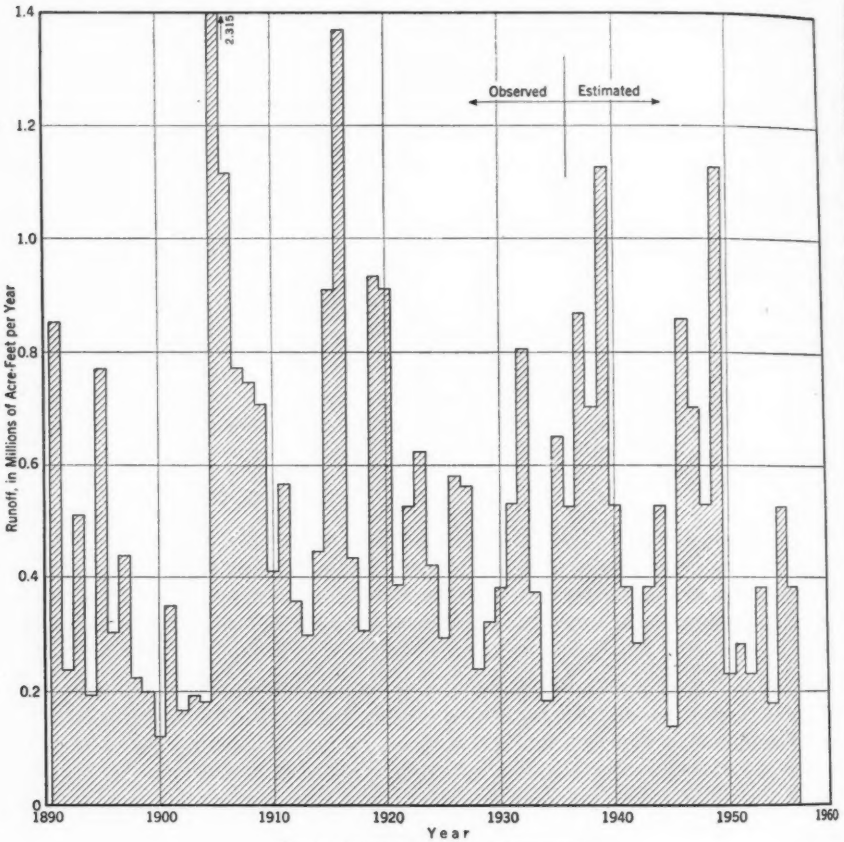

FIG. 2.—HYDROGRAPH AT CHRYSOTILE

solution that would be most economical for the sequence of flows that will occur in the future. In fact, as no evidence exists to show that 2-year or 3-year sequences have been repeated, it may be stated that a solution based on a mass graph of past runoff could not give a result which could be stated to be most economical for the years to come.

Although the duration-curve method avoids this dependence on past sequence, and treats the data without regard to chronology, the problem of economical storage requirements thereby is made impossible of solution, due to the fact that storage requirements are a function of sequence. As an illustration, a period of 10 years with wet and dry years alternating would require only a small reservoir for regulation, whereas the same flows, arranged in a sequence

so that all wet years come first, followed by all dry years, would require a much larger reservoir for regulation; yet, in both cases, these data would give the same result when analyzed by duration-curve methods.

In this analysis, a hydrograph was constructed for the next 20 years based on certain requirements of probability and sequence derived from past records. This probable future hydrograph was then analyzed by mass diagram methods to determine economical storage requirements.

As has been stated elsewhere,[3] it is often convenient to plot the duration curve with the ordinate expressed as a percentage of the mean flow rather than absolute values. In most cases, the scale is selected so that the percentages of mean flow are equal divisions on the vertical scale. In this case, to avoid the computations necessary in reducing each flow to a percentage of the mean flow, the plot is made with absolute quantities, and the percentage of mean flows determined for those particular points desired in the study. The mean flow having been computed to be 536,000 acre-ft per year, Fig. 3 then shows that for 97.5% of the time the flow equaled 130,000 acre-ft per year, or 0.24 of the mean flow.

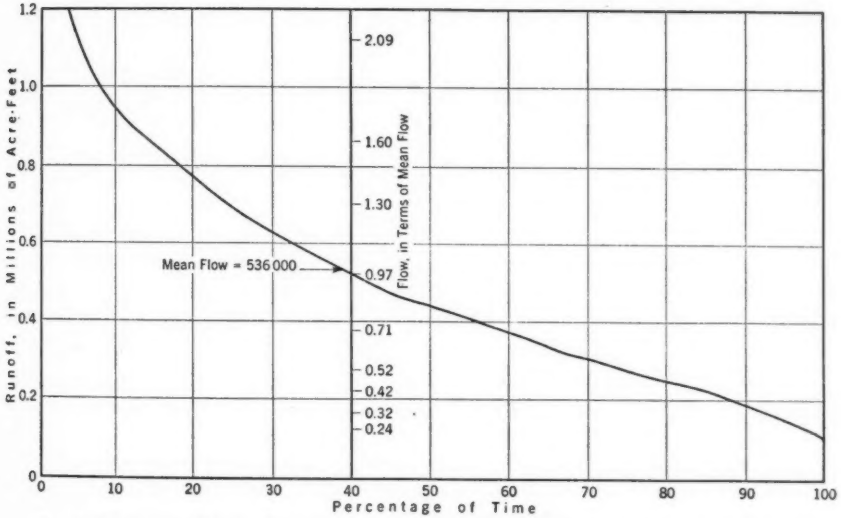

FIG. 3.—DURATION CURVE (CURVE APPLIES TO THE GAGING STATION AT CHRYSOTILE, ARIZ.)

With the flow established as a percentage of the mean flow, it becomes possible to compute the probable monthly flows, in terms of the mean monthly flows, for the various percentages of time.

The values of the probable monthly and yearly total runoff for various percentages of time are shown in Table 1. For the 20-year period the runoffs, in acre-feet, were:

Total probable	10,338,000
Average observed	10,800,000

[3] *Transactions*, Am. Soc. C. E., Vol. 99 (1934), p. 1220.

The close agreement is not surprising, as the duration curve was integrated from the frequency curve, and differentiation of the duration curve must of necessity give the frequency curve.

TABLE 1.—PROBABLE RUNOFF (AT CHRYSOTILE), IN THOUSANDS OF ACRE-FEET

Month	Mean flow	Type of Year								
		Flood	Very wet	Wet	Normal	Below normal	Dry	Very dry	Drought	Extreme drought
Percentage of time*	....	5	15	25	40	60	75	85	92.5	97.5
Flow in terms of mean flow	1.00	2.09	1.60	1.30	0.97	0.71	0.52	0.42	0.32	0.24
Occurrences in twenty years	....	2	2	2	4	4	2	2	1	1
January	46.7	97.6	74.7	60.7	45.3	33.3	24.3	19.6	15.0	11.2
February	74.8	156.5	119.6	97.2	72.6	53.1	38.9	31.4	24.0	17.9
March	105.0	219.5	168.0	136.5	101.8	74.6	54.6	44.1	33.6	25.2
April	95.2	199.2	152.0	124.0	92.4	67.6	49.5	40.0	30.5	22.9
May	48.1	100.5	76.9	62.5	46.6	34.2	25.0	20.2	15.4	11.5
June	18.7	39.1	29.9	24.3	18.1	13.3	9.7	7.9	6.0	4.5
July	21.1	44.1	33.7	27.4	20.5	14.5	11.0	8.9	6.8	5.1
August	31.2	65.2	49.9	40.5	30.6	22.2	16.2	13.1	10.0	7.7
September	24.5	51.2	39.2	31.8	23.8	17.4	12.7	10.3	7.9	5.9
October	17.8	37.2	28.4	23.1	17.3	12.6	9.3	7.4	5.7	4.3
November	24.2	50.6	38.7	31.4	23.5	17.2	12.6	10.2	7.7	5.8
December	32.9	68.8	52.8	42.8	31.9	22.9	17.1	13.8	10.5	7.9
Total	536.0	1,129.5	863.8	702.2	524.4	382.9	280.9	226.9	173.1	129.9

* Percentage of time in which a given flood type was exceeded.

TYPES OF YEARS

It is generally noted that all of the twelve months in a year follow the year type with respect to volume of water. In a wet year, of greater than average runoff, each of the individual months will be above average runoff for the month; and for the dry years, each individual month will be relatively dry. Thus, it follows, in general, that a wet year is composed of wet months, or, the infrequently occurring floods of a January most probably will be followed by the equally infrequently occurring floods of February.

For this study an arbitrary classification of year types was made: The infrequent "flood" year was exceeded only 5% of the time; the "very wet" year was exceeded 15% of the time; the "wet" year was exceeded 25% of the time; the "normal" year was exceeded 40% of the time; and so on, as shown by the headings of Table 1.

Again referring to Table 1, if the percentage of time is taken as a corresponding ratio to a given number of years (20 years, for example) the number of occurrences for each year type during the 20-year period can be obtained. If the runoff for each of the 20 years thus obtained were noted on a separate slip of paper, with the monthly values and total for the year, and the slips were drawn at random and noted in sequence, a hydrograph would be constructed which could be reduced to identical probability and duration curves of the observed runoff.

The number of different sequences which might be obtained by such a method is enormous,[4] but in every case the hydrograph so constructed would reduce to probability and duration curves identical to those obtained from observed data. Two or three trials will indicate the great range of sequence which may be obtained, and will show clearly that a mass diagram obtained from two different sequences may give widely different results, in spite of the fact that the hydrograph so constructed will meet conditions imposed, of similarity in probability to the observed runoff.

Although at present it does not seem possible to determine which of the many sequences will be the one that will be observed in the future, there are other data which will limit the number and establish certain conditions, thereby reducing the number of sequences possible and defining a limit of error. One limiting condition is imposed by the probability of prolonged drought. Although the driest year of record is 1900, with a runoff of 119,700 acre-ft, the driest 2-consecutive-year period is not twice the driest year, but a value slightly greater. A duration curve of 2-consecutive-year periods is shown in Fig. 4, from which it may be seen that an additional limiting factor is added to the probable future hydrograph: The minimum flow for any 2-consecutive-year period must equal or exceed 400,000 acre-ft.

Applied to the data deduced for the probable future hydrograph, this requires that the "drought" and "extreme drought" types of years be separated in the sequence, because the sum of these 2 years, if occurring in consecutive sequence, would be less than 400,000 acre-ft.

Although this condition of separation for two particular years will not reduce the large number of possible sequences very much, it does reduce the possible error by a large amount, because the critical periods of any project depending on river flow are the drought years. Similarly, the 3-year sequences are found to equal or exceed 700,000 acre-ft, imposing another limitation on the sequences of the probable future hydrograph.

In addition to the foregoing items, which may be used to limit the number of possible sequences in the probable future hydrograph, there is apparently a large field open for investigation in the realm of direct prediction of future flows.

Future Weather

In 1932, the writer analyzed[5] some of the long-term trends in future weather, based on extrapolation of cycles observed in long-term records obtained from other sources. Since that time, and, perhaps, primarily due to the necessity of accurate knowledge of weather conditions as applied to aeronautics, considerable research has been done on weather trends and predictions. As is well known, commercial airlines are now able to predict weather a few days in advance with a high degree of accuracy.

Since 1932 George F. McEwen, of the Scripps Institution of Oceanography, has furnished interested parties with predictions for 1 year in the future, with a remarkable agreement between predicted and observed weather.

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[4] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 626.

[5] "Weather Records Projected into the Future," by John Girand, *Civil Engineering*, February, 1932, p. 76.

Although most projects are bonded for 20-year or 30-year periods, the first 5 or 10 are the most important, as success or failure is usually determined within that time. If, then, the trend of the weather for the next 10 years could be

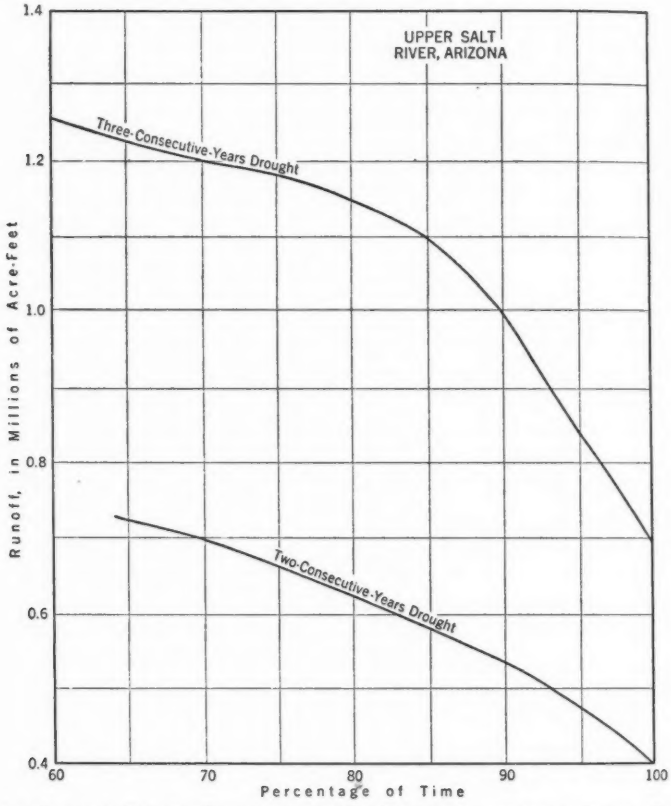

FIG. 4.—DURATION CURVES; DROUGHT SEQUENCE (CURVES APPLY TO THE GAGING STATION AT CHRYSOTILE, ARIZ.)

obtained with a reasonable degree of accuracy, and if the analysis would also show the worst condition which might occur in any length of time, the economic feasibility of the project could be determined. Accordingly, the probable future hydrograph should be constructed with particular emphasis on the next 10 years, and on the worst possible years.

LONG-RANGE FORECASTING

It was necessary for the writer to obtain data from other sources covering long periods of time to detect long-term variations in weather. The 12-year record of the Chrysotile gage, although extended to 40 years by correlation with other gages, was still too short to do more than indicate possible trends.

Fortunately, other long-term records were available. The observed rainfall observations at Ft. Apache, a town centrally located on the watershed, covered

a period of 60 years. An analysis of this and other rainfall records discloses a "cycle" which alternates in approximately 11 years. The discovery of the 11-year cycle in weather records opened the field of solar activity with respect to weather, as it had been known for many years that sunspot activity and related solar phenomena recurred in 11-year periods.

For example, H. T. Stetson has written:[6]

> "From a careful study of precipitation records selected over the globe [Clayton] has mapped the world into regions which show greater rainfall when sunspots are most numerous and regions where rainfall is actually deficient at sunspot maxima. While the north Atlantic shows 10 to 20 percent more precipitation in years of greater sunspots, the eastern half of the United States is in the region where the rainfall is actually less during maximum activity on the sun."

Another long-term record was available. Samples of tree rings taken at Cibecue, a town about 35 miles from the Ft. Apache rainfall gaging station, had been measured and recorded.[7] The tree-ring weather record is based on the fact that the annual ring of trees will show greater growth in wet years than in dry years, and by measuring the width and correlating with observed rainfall or runoff, a factor can be obtained which will allow the record to be extended back to the early years of the tree. The lower degree of accuracy of such measures is compensated for by the very long period of time covered by the record, 400-year to 500-year records being common, and, in rare cases (California Sequoias), records of 2,000 to 3,000 years may be obtained.

The trees sampled at Cibecue, being relatively young, only yielded a 269-year record; but a general record of Arizona has been developed for the past 1,200 years, the latter years of which agreed with the Cibecue record with sufficient accuracy to allow substitution.

The first interesting fact disclosed by this 1,200-year record is the recurrence of a 5-year drought sequence at 300-year intervals. The writer called attention to this phenomenon in 1932:[5]

> "An examination of the 1,200-year record previously referred to discloses periods of intense drought in the years near 700, 1000, 1300, 1600, and 1900 * * *.
>
> "Residents of the district under observation in Arizona well remember the drought of 1900–1905. For example, Mormon Lake, the largest natural body of water in the state, was completely evaporated, for the only time in its existence, so far as is known."

The 1,200-year record does not show any 5-year droughts such as this one occurring except at intervals of 300 years, indicating a 300-year cycle of this particular drought. Accordingly, the 1900–1905 sequence was neglected in all computations in which the sequence of dry years was involved in the probable future hydrograph for the next 20 years.

The second interesting fact disclosed by the 1,200-year record is the change in general climate during the Middle Ages. Archeologists had long wondered

[6] "Sunspots and Their Effects," by H. T. Stetson, McGraw-Hill Book Co., Inc., New York, N. Y.
[7] "Climatic Cycles and Tree Growth," by A. E. Douglass, Carnegie Institution, Washington, D. C.

why the Indian pueblos of the Painted Desert in Arizona had been abandoned, but it was not until the 1,200-year weather record was developed that a satisfactory answer could be given. The record shows, clearly, a general decline in precipitation over a long period of years, which forced the Indians to abandon an area which became unsuitable for agricultural pursuits.

Such a general decline for the present time is not observed in the last century of the record, indicating that the project will not be subject to any conditions other than those recognized as normal for the past century. Should such a condition start in 1940, the past record would seem to indicate that it would be many years before the project would have to be abandoned.

The 11-year cycle in tree-ring growth and its paralleling with the 11-year sunspot activity cycle is well known. In the 1,200-year record the parallel follows closely, even to the period 1645–1715, when there were no sunspots.

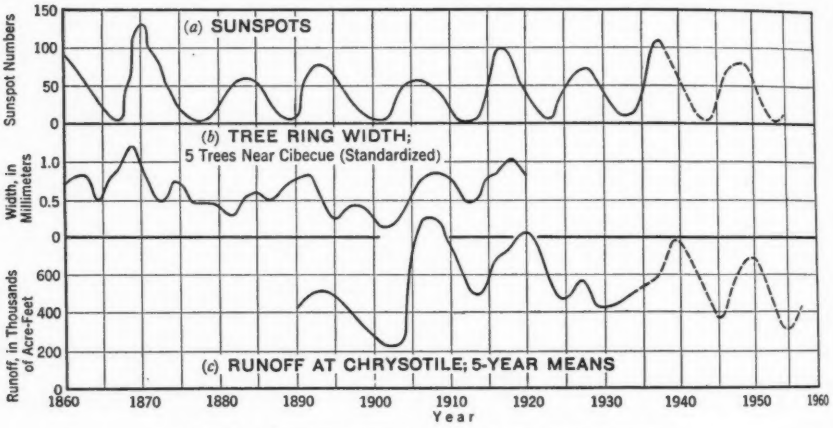

FIG. 5.—WEATHER TRENDS; UPPER SALT RIVER, ARIZONA

The definite cyclic trend of the sunspot cycle, as shown in Fig. 5, has been applied to tree rings, and correlated with the runoff on the Upper Salt River with such reasonable agreement as to allow a similar extrapolation to be made for the next 20 years. Although it is admitted that predictions of future sunspot maxima and minima are not always accurate, they do give a general trend of weather, which, when used in conjunction with the probable runoff as obtained from duration curves, will allow the construction of a probable future hydrograph which should represent conditions more closely than a sequence of flows drawn at random.

THE PROBABLE FUTURE HYDROGRAPH

From the data developed in the foregoing paragraphs, it is shown that the hydrograph of the future should meet the following conditions:

(*a*) Probability and duration curves derived from the future hydrograph must show agreement with the probability and duration curves derived from observed runoff;

(*b*) The arithmetic average of the 20-year period must agree with averages of observed records;

(*c*) The minimum 2-year and 3-year drought sequence must exceed 400,000 and 700,000 acre-ft respectively; and

(*d*) The general trend (a curve drawn through 5-year means) must follow the curve of sunspot activity. With these conditions in mind, the probable runoff, in acre-feet, at Chrysotile, was estimated as follows (the first three values being actual observations):

Year	Probable runoff, in acre-feet	Year	Probable runoff, in acre-feet
1933	370,000	1945	129,000
1934	178,000	1946	860,000
1935	653,000	1947	700,000
1936	580,000	1948	525,000
1937	830,000	1949	1,130,000
1938	700,000	1950	225,000
1939	1,130,000	1951	280,000
1940	525,000	1952	225,000
1941	380,000	1953	380,000
1942	280,000	1954	173,000
1943	380,000	1955	525,000
1944	525,000	1956	380,000

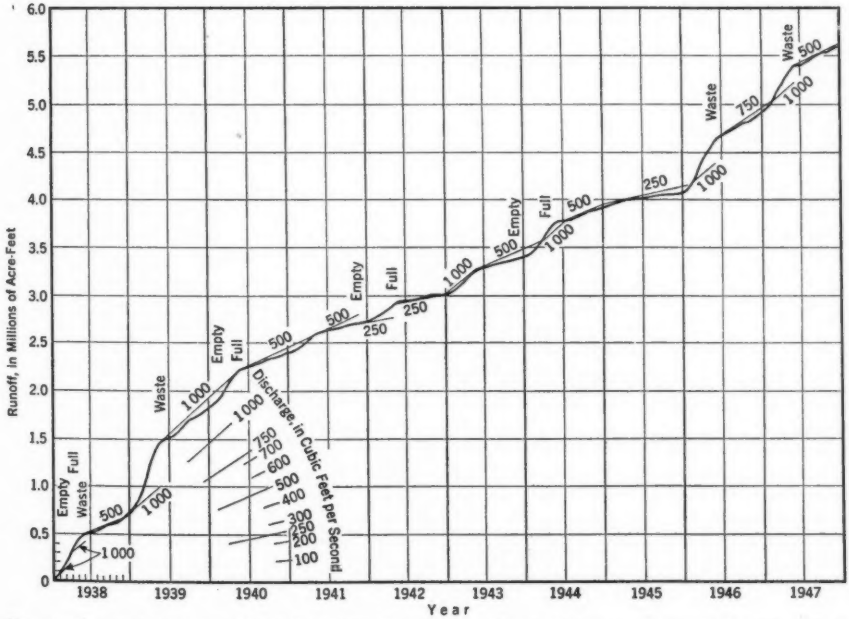

FIG. 6.—PROBABLE FUTURE MASS DIAGRAM; UPPER SALT RIVER, ARIZONA. (CURVES APPLY TO THE GAGING STATION AT CHRYSOTILE, ARIZ.)

The probable future hydrograph, drawn from the foregoing estimates, is shown in Fig. 2.

It is admitted that other sequences of the years could be made which would meet the foregoing conditions; however, it is believed that such other sequences would not show great differences in actual operation of the project, bearing in mind that the large floods are lost over the spillways, and the periods of drought are the most important in the operation of the project.

From the probable future hydrograph, the economical storage requirements and power output were estimated by the mass diagram method. Fig 6 shows the conditions which obtained for a reservoir of 100,000 acre-ft.

Conclusions

The basic idea adopted for this analysis—that future flows are not entirely fortuitous—should not be a concept that is foreign to engineers when practically every other phenomenon met by the profession is a result of a definite cause. River runoff is a result of rainfall, which, in turn, is a result of evaporation by solar radiation. Although, admittedly, the knowledge of the laws governing rainfall and runoff is now limited, and the fact that rainfall and runoff data have been forced into skew probability curves indicate fortuity, fundamentally, there should be a reason for the sequence of flows which are observed.

Thus far, a combination of flows obtained by probability methods, arranged in such sequence as to satisfy what little is known of cyclic phenomena, may be expected to give a closer approximation to the truth than an analysis based on probability methods alone.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

ANALYSIS OF LEGAL CONCEPTS OF SUBFLOW AND PERCOLATING WATERS

By C. F. Tolman [1] and Amy C. Stipp [2]

Synopsis

In this paper are presented some of the problems confronting the courts in attempting to determine types of ground-water occurrence according to the current legal classification of subsurface waters, which calls for differentiation between ground water flowing in definite subsurface water courses and so-called "percolating waters."

The complexity of a problem involving property rights to unseen, slowly percolating ground water is appreciated. The writers are not qualified to discuss the legal questions involved. However, the attempt is made to analyze, from the viewpoint of the ground-water hydrologist, some of the erroneous hydrologic concepts involved in the present legal classification of underground water and in past rulings following this classification. Elementary principles of hydrology governing the occurrence and movements of free ground water are presented to explain the fallacies in the legal concepts of "subsurface stream flow" and "percolating waters." For simplicity, principles of artesian flow, although of primary importance where confined water transmits pressure, are not discussed in this brief paper.

The writers appreciate that much of the material contained herein has received comprehensive treatment in a paper by Harold Conkling,[3] M. Am. Soc. C. E. However, it is believed that the subject warrants further independent treatment.

It should be noted that more than a quarter of a century has elapsed since publication of the legal textbooks cited in this paper.[4] In recent years much

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by **April 15, 1940.**

[1] Prof. of Economic Geology, Emeritus, Stanford Univ., Stanford University, Calif.

[2] Tucson, Ariz.

[3] "Administrative Control of Underground Water; Physical and Legal Aspects," by Harold Conkling, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 753.

[4] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912; also "Water Rights in the Western States," 3d Ed., by S. C. Wiel, Bancroft-Whitney Co., 1911.

progress has been made in the development of the science of ground-water hydrology under the able leadership of O. E. Meinzer and his staff of hydrologists in the U. S. Geological Survey.[5]

It is believed that a classification of subsurface waters for legal purposes based on well-established hydrologic principles would do much to correct the present confused status of ground-water law and the unsound concepts regarding ground-water occurrence and motions which have been handed down, in part at least, from a period prior to the development of the science of ground-water hydrology.

INTRODUCTION

Agriculture, mining, manufacturing, and even human existence depend upon the efficient utilization of ground-water supplies where severe deficiency in surface water exists. Development of the vast ground-water resources of the semiarid regions of the United States is subject to the establishment of legal rights to water supply; hence, the application of ground-water law affects the material welfare of all who are dependent upon industries using large quantities of water where surface-water supply is insufficient.

For legal purposes subsurface water (without considering artesian water) is divided into two main classes, based on a difference (erroneously assumed) in method of transmission through the ground, namely: (1) "Percolating waters"; and (2) "subsurface water courses." The term "percolating waters" has been defined in numerous ways and applied to embrace ground-water occurrence under varying hydrologic conditions. By some courts the term is restricted to "diffused waters" in privately owned lands; and by others it has been extended to include all subsurface water not proved to occur in "underground water courses" with definite subsurface "bed and banks." Likewise, the term "subsurface water course" has no fixed meaning and many conflicting interpretations have been offered by courts of law. It is not surprising that this lack of a sound fundamental legal classification of subsurface waters has resulted in confused ideas regarding the occurrence and motions of water underground.

In many jurisdictions rights to ground-water supply still depend upon the court's differentiation of the two legal "classes" of subsurface water. Some jurisdictions grant riparian or appropriative rights only for ground water in "subsurface water courses," and the so-called "percolating waters" belong exclusively to the owner of the land. Other states have recognized that an attempt to make such distinction is impracticable from both the economic and hydrologic viewpoints.

This question of differentiation, then, is of paramount importance in legal proceedings. For this reason, and because many curious and erroneous notions have arisen in the attempt to separate ground water in subterranean "water

[5] See especially *U. S. Geological Survey Water-Supply Papers: No. 494*, "Outline of Ground-Water Hydrology," by O. E. Meinzer, 1923; *No. 489*, "The Occurrence of Ground Water in the United States," with a discussion of principles, by O. E. Meinzer, 1923; and *No. 638-C*, "Methods of Estimating Ground-Water Supplies," by O. E. Meinzer.

courses" from "percolating waters," discussion in this paper is limited to two types of ground-water occurrence: (1) Ground water moving as a concentrated current bounded by relatively impervious subsurface "bed and banks"; and (2) the general body of ground water adjacent to the subsurface stream.

Other typical occurrences of ground water are mentioned briefly in order that the reader may appreciate that this discussion is strictly limited in scope. Part I, a brief review of principles underlying the occurrence of ground water in "subsurface water courses" and the interrelation of surface flow, subflow, and "percolating waters" under various hydrologic conditions, is presented in order that the analysis of legal rulings in Part II may be understood by readers not familiar with these fundamental natural laws. The necessity of a knowledge of the elementary principles of ground-water hydrology (see Appendix, Item 1) as a basis for legal rulings regulating the use of subsurface water is emphasized, and it is hoped that the simplicity of the general principles outlined herein may encourage lawyers to review the present-day science of ground water. Those versed in ground-water hydrology may be interested chiefly in Part II and the comparison between man-made laws regulating subsurface-water supplies and the laws of nature governing occurrence and movements of water underground.

PART I.—OCCURRENCE AND MOVEMENTS OF WATER UNDERGROUND

The term subsurface water includes all types of water occurring below ground surface. The water of the zone of saturation is ground water. Suspended water (see Appendix, Item 2) occurs in the undersaturated zone above the water table (ground-water level) and some of this water (gravity water) is in transit to the ground-water body. Although the efficient osmotic processes of plant roots can extract moisture from the soil, man can divert and utilize only ground water.

The zone of suspended water (zone of aeration) always exists above a body of ground water filling alluvial material unless saturation extends to ground surface. Surface water seeps slowly to the water table at a rate depending largely upon the permeability of the formations through which it passes. Gravity water (influent seepage) descends vertically over films of water attached to granular materials in the zone of suspended water.

Percolation.—Percolation is curvilinear flow around the grains and through the interconnected pores of saturated pervious materials. In granular materials (chiefly alluvial deposits) interstices are interconnected in all directions and free ground water (see Appendix, Item 3) moves in the direction of the slope of the water table. The rate of percolation of free ground water varies directly with water-table slope or hydraulic gradient and permeability of the water-conducting material.

The scientific meaning of the term percolation should not be confused with the legal application of the term "percolating waters" as distinguished from flow in subsurface water courses. "Subsurface streams," "percolating waters," and "artesian waters," as defined by lawyers, all move by percolation in interconnected openings of capillary size.

Influent and Effluent Conditions.—Ground water is supplied by influent seepage from rainfall, surface flow, impounded surface water, and from irrigation water and is discharged by effluent seepage, springs, and by plant roots if they reach the capillary fringe above the water table (see Appendix, Item 4).

The reactions between surface and subsurface "stream flow" and "percolating waters" depend upon whether influent or effluent water-table conditions exist. To illustrate the phenomena accompanying influent and effluent seepage one may start with the equilibrium that exists where influent conditions merge into effluent conditions and vice versa. The surface stream neither receives water from, nor loses water to, the subsurface water body. In such a case the stream level and the water table are shown as a horizontal line in a cross section at right angles to surface flow (Fig. 1), and there is no movement of ground water toward or away from the stream.

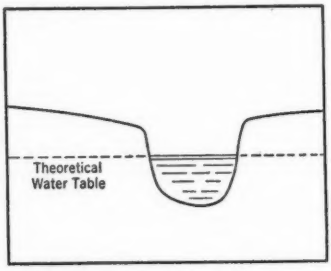

FIG. 1.—CONDITION OF STATIC EQUILIBRIUM BETWEEN WATER TABLE AND SURFACE STREAM

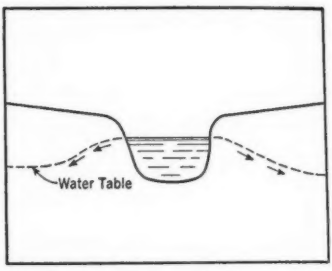

FIG. 2.—GROUND-WATER MOUND IN CONTACT WITH STREAM FLOW

If the ground-water level in the alluvial material is lowered by pumping or by natural causes, the surface stream loses water and becomes influent, the water table slopes away from the stream, and a ground-water mound is developed in contact with surface flow. In Fig. 2 the stream and the water table are influent. The direction of the ground-water movement is shown by arrows. If the ground-water level is lowered below stream bed, the ground-water mound is not in contact with surface flow. The material between the mound and the stream bed is not completely saturated and a column of influent seepage transmits the leakage from stream bed to the ground-water mound. In Fig. 3 a

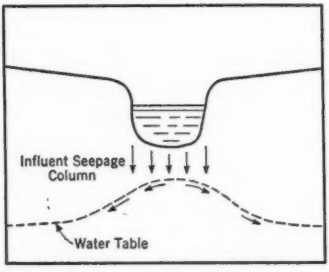

FIG. 3.—GROUND-WATER MOUND NOT IN CONTACT WITH SURFACE STREAM

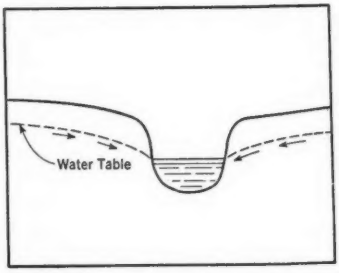

FIG. 4.—GROUND-WATER TRENCH WHICH SUPPLIES WATER TO SURFACE STREAM AND INDICATES EFFLUENT CONDITION

column of influent seepage is developed below surface flow. After the mound has lost contact with the surface stream, further lowering of the water table does not increase the rate of influent seepage.

Apparently lawyers do not generally appreciate the fact that stream flow occurs over nonsaturated gravels through which water seeps from the surface to the water table. The assertion is made that "the water from the surface stream must necessarily fill the loose, porous material of its bed to the point of complete saturation before there can be any surface flow." Often the stream bed is rendered relatively impervious by silt deposited with receding flood flows or by chemical cementation, and subflow occurs only at some distance below the surface stream, supplied by slow influent seepage. In general, materials below stream bed are not uniformly pervious and such conditions do not favor development of water-table mounds in contact with surface flow. It is not uncommon to find the water table at considerable depth below a surface stream, especially in the lower reaches of a desert stream just before the surface flow disappears into the stream gravels.

If the stream level is lowered below the water table by diversion of surface flow or by natural causes, a ground-water or water-table trench is formed which contacts surface flow at stream level. Ground water is percolating toward the stream and supplying it with water, and the stream level is adjusted to the quantity of effluent seepage. (In Fig. 4 the arrows indicate the ground-water movement toward the stream.) Diversion of stream flow increases the water-table slope toward the stream and increases effluent seepage. The legal dictum that a surface stream is "supported" by subflow is applicable only under effluent conditions.

Influent and Effluent Conditions as Shown by Ground-Water Contours.—The most satisfactory method of determining the direction of ground-water flow is by the construction of a water-table contour map. Direction of percolation is at right angles to the ground-water contours. Influent and effluent conditions are given graphic representation on such maps. Effluent conditions along a

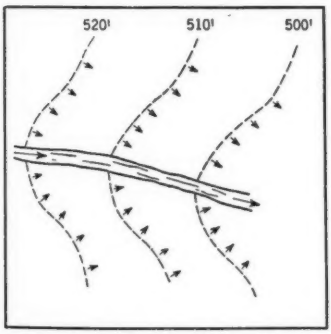

FIG. 5.—CONTOUR MAP OF WATER-TABLE TRENCH OF AN EFFLUENT STREAM

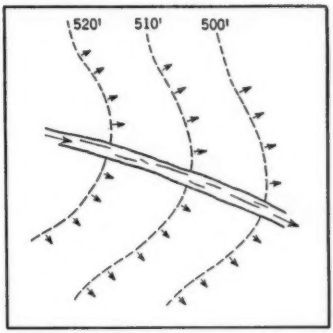

FIG. 6.—INFLUENT GROUND-WATER CONTOURS SHOWING WATER-TABLE MOUND OR RIDGE OF INFLUENT STREAM

stream course are shown by an upstream bend of the contours (as in Fig. 5), indicating the size and shape of the ground-water trench.

Influent conditions and the development of a ground-water mound or ridge are indicated in Fig. 6 by a downstream bend of water-table contours.

Effects on Surface Flow of Pumping from Influent and Effluent Subflow.—Under influent conditions draft on subflow by pumping does not increase stream seepage loss unless the slope of the water table in contact with the surface stream is increased. This occurs when the "cone of pumping depression" (see Appendix, Item 5) contacts surface flow. If the water table is lowered so that it loses contact with the surface stream, increased draft accompanied by further lowering of the water table does not increase the rate of influent stream seepage.

Under effluent conditions, pumping draws into the well ground water percolating toward the surface stream. The effect on stream flow depends on the ratio of effluent seepage thus diverted to the total effluent seepage feeding the stream. Diversion of surface flow will increase effluent seepage into the stream, at least temporarily, and will lower the water table adjacent to the stream by steepening the ground-water trench.

The attention directed by the courts to the effects on surface flow produced by pumping from a subsurface stream makes it advisable to discuss these pumping effects further. Pumping from a "stream" of ground water or from any free ground-water body produces a cone-shaped depression of the water table around the well. This cone of pumping depression expands until it intercepts a quantity of ground-water flow equal to the amount pumped. Thereafter the cone of depression remains approximately constant, but the effect of interception of ground-water flow (area of diversion) spreads out in the downstream direction of water-table slope. It has been stated in legal proceedings that the ground-water level responds to pumping in the same way as the water level of a surface reservoir, which, of course, is not in agreement with hydraulic laws.

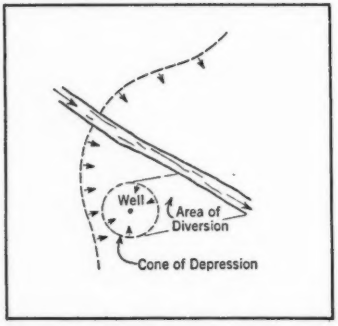

FIG. 7.—EFFLUENT WATER TABLE AND CONE OF PUMPING DEPRESSION SURROUNDING WELL

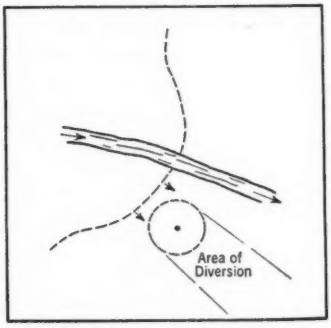

FIG. 8.—UNDER INFLUENT CONDITIONS THE CONE OF PUMPING DEPRESSION INTERCEPTS GROUND WATER MOVING AWAY FROM THE SURFACE STREAM AND AREA OF DIVERSION EXTENDS IN DIRECTION OF GROUND-WATER FLOW

Fig. 7 represents effluent conditions under which the effects of diversion of ground water will reach the surface stream even if the cone of depression does not. If the cone of depression expands as a result of overpumping the natural

supply and contacts the surface stream, water is drawn directly from the stream in addition to the subflow intercepted.

Under influent conditions (Fig. 8) if the cone of pumping depression does not reach the surface stream, no pumping effect will be registered by surface flow as far downstream as influent conditions extend.

If the cone of pumping depression extends under the surface stream, water is evacuated from the materials below stream bed within the cone of depression and an influent seepage column is developed below surface flow. Regardless of whether influent or effluent conditions existed prior to the development of such a cone of depression below stream bed, the pumping well is supplied by diversion of subflow plus water furnished by the induced influent seepage from the surface stream.

It is fairly common that the flood-flow channel of streams in arid regions is a mile or more in width. If subflow occurs in a corresponding subsurface channel, it is evident that, under extreme conditions, if pumping should affect the entire width of the subsurface stream, much more water would be abstracted from the ground-water "stream" than from the relatively small cross section of surface flow, even if all surface flow is diverted by such pumping. In general, as the width and depth of the subsurface stream are usually much greater than the surface stream, the quantity of subflow diverted by the pump is larger than the quantity of surface flow abstracted.

The legal "test" for subflow, discussed in Part II, and the prevailing rule that "there cannot be any abstraction of the water of the underflow without abstracting a corresponding amount from the surface stream" are inconsistent with these principles.

Downstream Effects of Diversion Above a Change from Influent to Effluent Conditions.—Thus far this paper has been limited to reactions between surface flow and subflow within areas of influent or effluent water-table conditions. However, abstraction of ground water under influent conditions which may not affect influent stream flow may affect surface flow in a downstream area below a change from influent to effluent conditions. Diversion of subflow by pumping may so diminish ground water that it does not fill the sand and gravel channel where effluence occurred prior to pumping. In the long desert rivers that carry surface flow throughout effluent sections of the stream course and are dry where influent conditions exist, diversion of either surface or subsurface flow may reduce subflow in downstream influent portions and may diminish both subflow and surface flow in downstream effluent stretches of the stream.

Geological Conditions Favoring "Subsurface Stream Flow."—Ground water in stream sands and gravels deposited in a canyon or valley cut in solid rock is the most definite example of a "subsurface stream." Such conditions commonly exist along the mountainous headwaters of the stream. The "bed and banks" of the subsurface water course are carved in bedrock. Surface flow depends upon the subsurface stream where the latter is effluent, and subflow depends on surface flow where the surface stream is losing water by influent seepage.

After leaving the mountain uplands, most streams of the arid and semiarid regions of western United States transect a body of valley fill. It is fairly common for the larger streams to cut channels a mile or more in width through the older alluvial fill. The outermost banks cut by the stream flow are the upper flood-plain terrace faces, and the subflow "banks" might be considered to be the subsurface continuation of these terrace faces. The stream-channel and flood-plain deposits, which the stream has laid down during late depositional stages, are usually more pervious than the older valley fill and, if so, the hydrologic behavior of surface and subsurface flow shows similarities to that in rock-bound channels.

Subflow Indicated by Contour Pattern.—The ground-water contour pattern represents water-table conditions, and spacing of contours indicates the gradient of percolating water. Characteristic patterns are developed by a subsurface stream where the bed and banks are impervious bedrock and the channel deposits are pervious. Similar but less definite patterns are shown by a subsurface stream with bed and banks cut in material less pervious than the stream-channel deposits but not impervious.

The stream-laid deposits in a rock-bound channel are charged with water during flood flow and a pronounced water-table ridge is developed. The water thus stored above the low-water stages is discharged into the stream with subsiding flood flow and develops a water-table trench. Also, a rock-bound subsurface water course is usually effluent in a contracting part of the bedrock channel and develops a water-table trench; it is influent where the subsurface channel expands and a water-table mound is formed. Where bedrock is impervious the ground-water contours terminate at the bedrock banks.

Summarizing, ground-water contour patterns show the following relations to impervious bedrock boundaries of a subsurface stream:

(1) The contours terminate at the bedrock banks;

(2) Within subsurface banks the contour pattern fluctuates from influent form during rising water stages to effluent shape during subsiding water stages; and

(3) A decrease in cross-sectional area of subsurface channel develops an effluent contour pattern, and an increase in cross-sectional area produces the influent pattern.

Comparing these contour patterns with patterns developed by a subsurface stream confined between banks of material less pervious than the stream-channel deposits but not impervious, one finds that the ground-water contours bend rather than terminate at the banks of the subsurface channel. When flood flow produces a pronounced mound in the subsurface stream, the water-table contours beyond the subsurface banks are also bent into a less pronounced influent shape, indicating slight influent movement from subflow into the older valley fill. Likewise, where a sharp ground-water trench is produced by drainage of saturated stream gravels during low-water stages of surface flow, the contours beyond the subsurface banks are drawn into the effluent form and slight effluent movement is developed from the older alluvium into the subsurface stream channel (see Figs. 9 and 10).

The boundaries of such a subsurface stream course are indicated by:

(1) A bend in ground-water contours at subsurface banks;

(2) Change of contour pattern with change from high-water to low-water stage of surface flow; and

(3) Changes in contour pattern with fluctuation in cross-sectional area of subsurface channel.

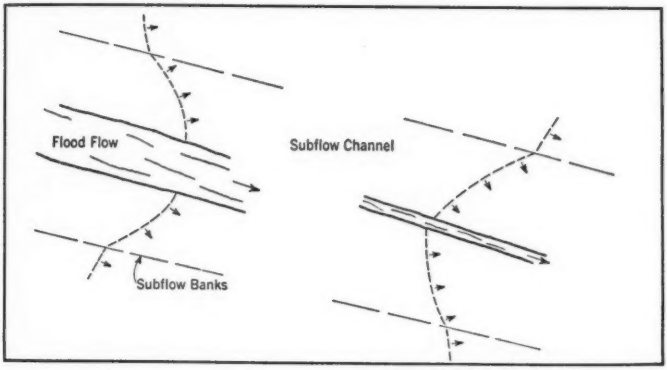

FIG. 9.—PRONOUNCED GROUND-WATER MOUND IN SUBFLOW CHANNEL PRODUCED BY FLOOD FLOW, WITH CONTOURS BEYOND LESS PERVIOUS BANKS MODIFIED INTO INFLUENT SHAPE BY GROUND-WATER MOVEMENT INTO OLDER ALLUVIUM

FIG. 10.—DRAINAGE OF STREAM GRAVELS WITH LOW-WATER SURFACE FLOW, PRODUCING PRONOUNCED TRENCH IN SUBFLOW CHANNEL WITH CONTOURS BEYOND SUBSURFACE BANKS DRAWN INTO EFFLUENT SHAPE DUE TO SLOW EFFLUENT MOVEMENT FROM OLDER ALLUVIUM

The interpretation of contour pattern as indicating the extent of subsurface flow has been emphasized because the water-table contour pattern is the simplest and most easily recognized criterion that can be thus applied. Unfortunately, water-table maps are seldom constructed to the scale and in the detail required for such interpretation. If physical conditions render the construction of special detailed maps difficult and costly, other methods of investigation may be applied. Some of these are mentioned without further comment:

1. Topographic: Recognition of subsurface stream course by topographic features, especially stream terraces.

2. Geologic: (*a*) Differentiation of older alluvium and recent stream deposits by means of samples taken at the surface and from wells; and (*b*) location of "bed and banks" cut in older alluvial deposits by subsurface projection of flood-plain terrace faces or by shafts and drifts to expose the subsurface banks.

3. Hydrologic: (*a*) Permeability tests of samples mentioned under Item 2(*a*); and (*b*) analysis of pumping tests on wells tapping subflow and wells drawing on "percolating waters" beyond subsurface banks.

Ground Water Not Occurring in Subsurface Water Courses.—West and south of the Rocky Mountains the main streams heading in the mountains traverse a series of structural valleys formed by the depression of bedrock between uplifted mountain ranges, not by stream erosion. The principal source of water

supply is mountain runoff collected by the upper reaches of the master stream and moving as surface and subsurface flow in the sands and gravels of the stream channel. Such subflow may be the principal ground-water supply of the region, and the best agricultural lands are situated on the flood-plain and terrace deposits. However, the notion that ground water must be concentrated in a restricted subsurface channel to constitute a supply sufficient for industrial or agricultural use, entertained by many well drillers and apparently accepted by some lawyers, is in error. There are many occurrences of ground water of great economic importance that cannot be classified as subsurface water courses. In permeable granular deposits ground water percolates through interconnected pores between grains throughout the body of the pervious material. In dense impermeable rocks it moves through the fracture systems. In lava containing large openings produced by flowage and fracture and in cavernous limestone ground water flows freely through large tubular openings. Each of such typical occurrences is distinctive and the type of openings carrying ground water should be determined before an attempt is made to interpret the interrelation between ground water and surface water.

In some areas ground-water supplies in lava and limestone are important; in others the only available ground water may occur in fractures. In some sections of the United States extensive confined (artesian) aquifers yield copious supplies. The alluvial deposits of the semiarid western United States contain large supplies of ground water that cannot be classed as subsurface stream flow, and where restricted subflow does occur it may be of minor importance.

The bedrock of large intermontane valleys or basins (see Appendix, Item 6) of southwestern United States is generally covered with a thick deposit of alluvium derived from the surrounding mountain ranges. The valley fill consists of coalescing alluvial fans and cones, and either the central lake or playa deposits of undrained basins or the sand and gravel deposits of the master stream which drains the basin. The clayey playa deposits do not yield large ground-water supplies, but the pervious channel deposits of axial streams usually carry subflow and are important water bearers.

Where the alluvial cones flanking these semiarid basins are large and the adjacent mountains furnish sufficient water, the principal subsurface supplies may be held in the alluvial cones. Where the stream draining the valley has not built up thick pervious channel deposits, there will be no large subflow associated with the surface stream in the center of the valley. The southern part of Owens Valley, California, is a good example of such conditions.

The lower or distal parts of alluvial cones may consist of fine, relatively impervious material which separates the ground water in the cones from that of the longitudinal valley. Alluvial cones also commonly contain artesian water. An understanding of the hydrologic characteristics of alluvial cones and of the principles governing the flow of artesian water[6] is essential for the solution of problems involving the effects of pumping from alluvial cones.

The foregoing condensed summary of a few elementary principles of ground-water hydrology offers a basis for understanding the geologic and hydrologic

[6] "Ground Water," by C. F. Tolman, Chapters XI and XII, McGraw-Hill Book Co., Inc., 1938.

problems involved in the determination of the type of ground-water occurrence and in interpretation of reactions between surface and subsurface water bodies. Legal decisions made with inadequate comprehension of these physical laws may not be applicable to the particular conditions existing in the area in controversy; nor can they result in efficient regulation of the ground-water resources. Some of the difficulties encountered by the courts because of lack of understanding of hydrologic principles are presented in Part II.

PART II.—ANALYSIS OF A FEW RULES OF LAW

Many states, especially those of the semiarid West, have abandoned the long-accepted doctrines of water law applicable to regions of plentiful surface water supply and have attempted to establish rules better adapted to physical conditions in areas of low rainfall and intermittent stream flow.

Because of keen competition for limited surface and subsurface water resources, numerous water-rights cases, involving many technical points of law, have come into the courts. It is not the purpose of this paper to discuss these technical legal questions but merely to analyze some of the legal concepts and rules regarding subsurface water and to compare them with the behavior of underground water as deduced from modern scientific study.

In the legal literature the idea has often been expressed that occurrence and movements of subsurface water are incomprehensible and unpredictable. In a textbook on water law published in 1912,[7] it is stated that it is difficult to prove the existence of a subsurface water course because "the physical laws governing underground water and its subterranean progress [are] changeable and uncontrollable in character and subject to secret and incomprehensible influences."

It is true, indeed, that an accurate determination of the quantity of ground water available may be impossible because of insufficient data, but in many cases the critical data can be collected by detailed investigations. In no case are the physical laws "irregular or unknowable"; nor is ground water subject to "secret and incomprehensible influences." The physical laws governing the movements of ground water are well established, and relatively simple engineering investigations will determine the direction of ground-water motion, and geological observation will determine the type of occurrence.

The practice of most of the law courts in the past has been to differentiate sharply between the occurrence of ground water in subsurface water courses and as "percolating waters," applying different rules to regulate the use of ground water in these two supposedly different types of occurrence. In most jurisdictions riparian or appropriative rights to ground water can be upheld only if its occurrence as a subsurface water course or "lake" can be established. Several states of the West, however, have extended rights of appropriation to occurrences of "percolating waters," and, in general, the trend seems to be toward application of equal rights to all bodies of subsurface water subject to development.

[7] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912, p. 2116.

As explained in Part I, all ground-water bodies in granular materials, whether in defined subsurface channels, artesian systems, or in extensive subsurface "reservoirs" (see Appendix, Item 7), are percolating, except for rare instances of more rapid turbulent flow where a steep hydraulic gradient is developed in coarse, sorted gravel. Since analysis of the legal concepts regarding "percolating waters" involves the more elementary principles of ground-water hydrology, this legal class will be presented first.

"Percolating Waters"

The most curious concept expressed in legal literature is that of "diffuse percolations," which have been described as "vagrant, wandering drops moved by gravity in any and every direction along the line of least resistance." These waters are supposed not to "contribute, or are not tributary, to the flow of any definite stream or body of surface or subterranean waters."

As a matter of fact, there is no "diffused water" below the water table. Ground water completely fills all the openings in the material through which it percolates. No type of movement in natural deposits, either of ground water or of water seeping from ground surface to the water table, is comparable to that described for "diffuse percolations." The general acceptance of this concept of diffusion of percolating water by jurists has given rise to erroneous hydrologic dicta regarding the relation of "percolating waters" to other types of ground-water occurrence.

According to legal terminology, "percolating waters" have been variously defined and described as:

> "* * * those oozing or percolating through the soil in varying quantities and uncertain directions; water flowing, seeping, or circulating in minute particles beneath the surface of the ground, without banks or defined channels, and whose course is invisible and unknown * * *."[8]
>
> "* * * those waters which slowly percolate or infiltrate their way through the sand, gravel, rock or soil, which do not then form a part of any body of water or the flow of any water course, surface or subterranean, but which may eventually find their way by force of gravity to some water course or other body of water, with whose waters they mingle, and thereby lose their identity as percolating waters."[9]

It is evident that none of these descriptions is a correct representation of either the legal application of the term or the scientific meaning. Percolating waters as defined in the preceding quotations might be interpreted as gravity water seeping through the zone of aeration. It is stated that these waters "do not * * * form a part of any body of water * * *" and therefore, by strict application of legal definition, could not be considered a part of the ground-water body. However, water in the undersaturated zone above the water table is not available for pumping or diversion and therefore need not be considered in a classification for the purpose of establishing rights to water supply. Obviously, the term "percolating waters" does not apply to gravity water in the undersaturated zone, although the term percolation has been used

[8] 55 A.L.R. 1388, and cases cited.

[9] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912, p. 2150.

synonymously with infiltration. In the legal sense, any occurrence of subsurface water not proved to qualify for more definite classification is considered as "percolating."

The old common-law rule governing the use of "percolating waters" is:[10]

> "* * * the water which is held by the soil is a portion of the soil itself, and belongs to the owner of the land * * * . This rule is not changed by the character of the material through which the water percolates * * * . So long as the water is in the condition of infiltration or percolation, it is a part of the soil * * * ."

The impractical character of this rule has been recognized and the majority of states have adopted new rulings. In Wrathall vs. Johnson[11] the question of appropriation and priority of use of subsurface water is discussed at length. The court states:

> "* * * Once a departure from or modification of the common-law rule has been made, as has been done in most of the states, we see no place for us to stop short of the rule of prior appropriation. The later cases in arid regions are tending strongly towards the doctrine of appropriation and priority of use as the surest safeguard of rights to use of water * * * ."

Subsurface Reservoir.—"Percolating waters tributary to a subsurface reservoir" are usually distinguished from "vagrant, wandering diffuse percolations." The legal concept of "tributary percolating waters" is described by Kinney:[12]

> "* * * those waters which percolate down through the gravel, porous rock, and soil from various directions until they reach some barrier of impervious rock, clay or other material, constructed either by Nature or by the hand of man, thereby forming catchment basins, and there these percolating waters, mingling with the rock, gravel, sand and soil, form what are called underground reservoirs, with the water at times rising nearly or quite to the surface of the earth * * * . These waters tributary to underground reservoirs percolate, it is true, but only in the sense that they form a vast mass of water confined in a basin filled with detritus, always moving slowly down to lower levels in the effort, in conformity with physical law, to attain a uniform level."

There is no natural differentiation between percolating water "tributary" to a subsurface reservoir and the ground water of the reservoir. The common legal view that "percolating waters" are in the process of "infiltration" is erroneous. The true character of the movement of subsurface water should be emphasized. Whereas gravity water seeps downward through granular materials of the zone of aeration, percolation of the free ground-water body is always in the direction of water-table slope and the entire percolating water body below the water table constitutes the subsurface reservoir.

The view that the ground-water reservoir may be considered a static water body has been generally accepted in the past in legal proceedings. In Hinton vs. Little[13] the court offered an objection to this concept, ruling that all ground water is percolating.

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[10] Gould vs. Eaton, III Cal. 639, 44 Pac. 319 (1896).

[11] 86 Utah 50, 40 Pac. (2) 755 (1935).

[12] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912, pp. 2169–2170.

[13] 50 Idaho 371, 296 Pac. 582 (1931).

> "* * * It appears to be now fairly well settled that all underground subterranean waters are percolating waters; that is, that there is more or less movement, both perpendicular and horizontal, through the earth and rocks. Therefore, whether underground waters move in a well-defined channel, either in a generally confined direction as to the points of the compass or spread out laterally, is merely a question of difference of degree * * * .
>
> "It would seem, therefore, that it is impossible to attempt to lay down one rule with regard to subterranean waters, existing more or less as a relatively stationary body of water under the ground, and subterranean waters in which there is a decided movement * * * ."

To compare any ground-water body to a static lake, as was done in City of Los Angeles vs. Hunter,[14] is not justifiable. The effects of pumping from a surface reservoir or lake are very different from those produced by pumping from a free ground-water body, where the cone of pumping depression lowers the water level only throughout an area sufficient to divert the quantity pumped and ground water is continuously percolating through the water-bearing material to replenish the supply.

The significance of the fact that ground water never occurs as a stationary water body should be stressed. Ordinarily, the subsurface reservoir is continuously receiving additions by influent seepage from rainfall and surface water bodies and is always discharging water by natural processes. In the subsurface reservoir ground water is percolating toward the discharge area; no static ground-water bodies are known to exist.

In actions at law subsurface water is presumed to be "percolating water" unless bed and banks of an underground "water course" are definitely established. The hydrologic problems involved in an attempt to distinguish between "subsurface stream flow" and "percolating waters" are discussed in the following sections.

Subsurface Water Course

According to legal definition a water course "consists of a bed, banks, sides, or walls, and a current of water. It is a living stream confined to a channel, usually flowing in a particular direction * * * ."[15]

> "A 'stream of water' has a defined channel. It has banks, and is very distinct from the percolations of subsurface water, which oozes in veins or filters through the earth's strata. An 'underground stream' of water differs from a surface stream only with respect to its location above or below the surface."[16]

The legal concept of the geologic and hydrologic conditions determining whether a ground-water body constitutes "percolating waters" or a subsurface water course is described in Miller vs. Bay Cities Water Company, as follows:[17]

> "* * * A water course must consist of bed, banks or sides and water, and the water must be flowing in a channel or course more or less defined * * * . Waters moving by force of gravity in a valley or basin of wide

[14] 156 Cal. 603, 105 Pac. 755 (1909).

[15] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912.

[16] Stoner vs. Patten, 132 Ga. 178, 63 S.E. 897 (1909).

[17] 157 Cal. 256, 107 Pac. 115, 27 L. R. A. (N. S.) 772 (1910).

> extent * * * and moving generally through the whole or through a large portion of the basin, along through the natural voids or interstices of the earth, composed of alluvial or other deposits, lying throughout the entire basin, and made up of loam, sand, gravel and bowlders, mixed together, and interspersed with irregular and broken strata or masses of clay or cemented sand and gravel, and lying in place as originally deposited by the forces of nature, do not constitute a water course, but are a part of the land, and belong to the owner of land as fully as any other constituent part of it * * * . But if such water does collect underground and flow in certain courses or channels through coarse, permeable material therein, where the existence and general course of the flowing or moving body of water can be easily determined, then the water so moving in such channels would constitute a water course, although not visible on the surface, and although the space through which the channel extends may be largely filled with the material through which the water flows."

It should be realized that, in the light of present-day knowledge, "the existence and general course of the flowing or moving body of water" can be determined as readily for the water percolating throughout an extensive "basin" or valley as for ground water flowing in a restricted subsurface channel. Whereas percolation of ground water below a surface stream is frequently restricted by fairly well defined "bed and banks," subflow often cannot be differentiated from the general ground-water body except as a water-table mound or trench below surface flow.

As a matter of fact, considerable difficulty has been encountered by the courts in delimiting subflow and in defining a subsurface water course. In City of Los Angeles vs. Pomeroy[18] the court defined the boundaries of a water course as follows:

> "* * * While a water course must have a bed and banks or sides, yet such bed may consist of any material which keeps the waters from penetrating below a certain depth, and such banks or sides may consist of any material which has the effect of confining the waters within circumscribed limits."

Every ground-water body, of course, meets the conditions of this ruling, as the limiting boundaries of any body of ground water may be defined more or less accurately. The extent of the subsurface reservoir, if not restricted to a subflow channel, is usually considered as limited to some topographic unit, such as the pervious area of a surface drainage basin or the slopes of an alluvial cone. Although the saturated water-conducting material is never uniformly permeable throughout and may be interrupted locally by lenses or layers of impervious material, the subsurface reservoir is considered to extend throughout the pervious alluvial material of the basin.

What constitutes a "known and defined" subsurface water course, then, depends upon the viewpoint of the court, and decisions have been based on various types of evidence.

In City of Los Angeles vs. Pomeroy[18] it was ruled that:

> "* * * 'defined' means a contracted and bounded channel, though the course of the stream may be undefined by human knowledge; and the

[18] 124 Cal. 597, 57 Pac. 585 (1899).

word 'known' refers to knowledge of the course of the stream by reasonable inference."

Numerous decisions have been based on the requirement that:

> "* * * without opening the ground by excavation, or having recourse to abstruse speculation of scientific persons, men of ordinary powers and attainments would know, or could with reasonable diligence ascertain, that the stream, when it emerges into light, comes from, and has flowed through, a defined subterranean channel."[19]

In Maricopa County Municipal Water Conservation District vs. Southwest Cotton Company[20] the rule applied was:

> "* * * there must be clear and convincing proof to the satisfaction of a reasonable man, not only that there are subterranean waters, but that such waters have definite bed, banks and current, and evidence must establish with reasonable certainty the location of such bed and banks; there must be certainty of location, as well as of existence of the stream * * *."

The rulings in this case are significant. The trial court accepted the geological evidence presented as conclusive that ground water occurred in a dependent subsurface water course, fed by the influent river. However, a higher court reversed the decision, ruling, in regard to proof of a water course, that it is insufficient in Arizona to base appropriation on evidence that

> "* * * somewhere beneath the surface * * * geological theory teaches us that there are subterranean channels * * *. The decree of the trial court, so far as it adjudges plaintiff a right of prior appropriation to water of any wells not in or immediately adjacent to the surface channel of the Agua Fria River, cannot be sustained on the ground that such water comes from either dependent or independent underground channels with known and definite banks * * *."
>
> "* * * There is not a scintilla of evidence in the record from which the ordinary man, or even the trained scientist, could point out definitely a specific place where any one of the so-called subterranean water courses begins, where it ends, or how far its banks extend."

It appears, therefore, that each court must establish a ruling based on its interpretation of existing geologic and hydrologic conditions, and if courts refuse to accept scientific evidence indicating natural conditions their decisions may not be well founded.

Subflow.—In the legal sense, "underflow, subflow, or undercurrent of a surface stream" is considered as "those waters which slowly find their way through the soil, sand, and gravel constituting the beds of streams, or the lands under and adjacent to the streams which are themselves a part of subsurface streams." By strict application of the conditions given in this definition, subflow could be considered only as the ground-water mound or ridge in contact with an influent stream or the ground water adjacent to an effluent stream.

The test applied to determine whether or not ground water constitutes subflow is:[20]

> "* * * there cannot be any abstraction of the water of the underflow without abstracting a corresponding amount from the surface stream, for

[19] Clinchfield Coal Corp. vs. Compton, 148 Va. 437, 139 S.E. 308 (1927).

[20] 39 Ariz. 65, 367, 4 Pac. (2) 369 (1931), 7 Pac. (2) 254 (1932).

> the reason that the water from the surface stream must necessarily fill the loose, porous material of its bed to the point of complete saturation before there can be any surface flow. Therefore the river bed must continue holding sufficient water to support the surface stream * * * for otherwise in drawing on the underground flow of the stream it will necessarily draw upon the water flowing on the surface."

Analyses of the definition of subflow and the determining test disclose numerous fallacies. Without going into detail, the following discussion should indicate some of the hydrologic principles involved.

Subflow as "a Part of the Surface Stream."—The waters of subflow are specifically qualified as being "a part of the surface stream." Effluent subflow or influent subflow, with ground-water mound in contact with surface flow, comply with this qualification. Obviously, effluent subflow, which is feeding the stream, may be considered to constitute a part of surface flow. An effluent ground-water body "supports" the surface stream, in that it contributes to surface flow.

A ground-water mound built up to surface stream level also might be considered a part of the surface stream, in so far as it is in contact with surface flow. A well that taps such a mound may abstract water directly from surface flow. However, where an influent seepage column occurs between surface flow and the water table, it is evident that subflow cannot be considered a part of the surface stream. In any case it cannot be said that a ground-water mound "supports" the surface stream, either in the sense that it furnishes a physical support or that it contributes to the supply of surface flow. On the contrary, the surface stream supplies all of the water of the mound.

It has been stated that diversion of subflow or influent "percolations" creates "an artificial draft upon the surface flow which draws down a part of it and weakens and injures the natural bed * * * ."[21] Numerous cases have been based on the theory that riparian owners or appropriators of surface flow are entitled to have the "percolating and seeping waters" remain undisturbed for the purpose of "supporting and maintaining" the water in the surface water body. A California court held that "a landowner cannot sink a well so near an irrigation ditch of another that the water supply would be taken from the ditch."[22] The principle involved, as stated in the legal brief, is "the right to the support to the water flowing in the ditch afforded by the seepage water escaping therefrom into the soil reasonably adjacent to the ditch."

In City of San Bernardino vs. City of Riverside[23] the following concept of "support" furnished by subflow is expressed:

> "* * * When a stream runs over porous material saturated with water, * * * the underground waters support the stream, either by upward or lateral pressure, or feed it directly * * * ."

This concept that "support" is generally afforded the surface stream by subflow is erroneous and is true only where effluent conditions exist. Obviously

[21] Vineland Irrigation District vs. Azusa Irrigation Co., 126 Cal. 486, 58 Pac. 1057, 46 L. R. A. 820 (1899).

[22] Lemm vs. Rutherford, 76 Cal. App. 455, 245 Pac. 225 (1926).

[23] 186 Cal. 7, 198 Pac. 784 (1921).

influent ground water furnishes no physical support to surface flow whether the ground-water mound is in contact or at some distance below the surface stream. Ground water percolates down the slopes of the water-table mound away from an influent stream and there is no "upward or lateral pressure" exerted on the water in the stream. Whether the stream is influent or effluent, the bed is not "weakened or injured" in any way by diversion of subflow.

The hydrologic test that

> "Drawing on the underground flow of the stream * * * will necessarily draw upon the waters flowing on the surface (because) the surface stream must necessarily fill the loose, porous material of its bed to the point of complete saturation before there can be any surface flow * * * ."

is not generally applicable. The erroneous assumption involved in the latter part of this statement is evident from previous discussion of the development of ground-water mounds by influent seepage. Pumping effects under influent and effluent conditions have also been treated in Part I. These will be reviewed briefly, however, from the standpoint of the test for subflow.

Influent and Effluent Conditions.—Surface flow responds to the test for subflow only under certain conditions of the water table with relation to surface flow. If subflow is effluent, the test is partly valid. Pumping will diminish the quantity of ground water reaching the surface stream throughout the parts of the stream course affected by the area of pumping influence (see Fig. 7). Reduction of effluent seepage feeding surface flow will be in direct proportion to the decrease in gradient of the water-table trench sloping toward the stream. Furthermore, if the cone of pumping depression reaches surface flow, water will be abstracted directly therefrom. In most cases reduction of surface flow produced by pumping subflow is small compared to the quantity abstracted from the more extensive ground-water body.

If subflow is influent, pumping therefrom will draw directly on surface flow only if the slope of the ground-water mound in contact with the stream is increased, which occurs if the cone of pumping depression extends to the surface stream. The quantity thereby withdrawn from surface flow is directly proportional to the increase in water-table slope away from the stream. As explained in Part I, the quantity thus directly diverted from surface flow is usually small compared to the quantity withdrawn from the subsurface stream.

In the common case in arid regions where the ground-water mound is not in contact with the surface stream, pumping from the influent ground-water body can have no effect on surface flow throughout the area in which stream flow is feeding water underground. Pumping from an influent ground-water body not in contact with surface flow does not increase the rate of stream seepage loss. It is not true in any case that pumping from subflow abstracts "a corresponding amount from the surface stream," although it may draw directly on surface flow.

Desert streams often carry surface flow only where ground water is forced toward the surface by contraction of bedrock channel, producing effluent seepage. Pumping of subflow under such conditions may prevent or reduce effluence downstream from the pumps. Furthermore, pumping of effluent subflow or diversion of surface flow will diminish influent subflow downstream

from such diversion. On such streams subflow and surface flow are so closely interrelated as to make any distinction between the two types of flow almost impossible.

Streams in less arid regions may carry surface water throughout their course. Where restricted to narrow bedrock channels, or where the channel is cut through relatively impervious valley fill, the flow may fluctuate from influent during flood-flow stages to effluent conditions with low-water stages. Various local hydrologic and geologic conditions, also, may cause a change in the relation of water table to surface flow. The surface streams of alluvial cones are usually influent near the apex of the cone and effluent toward the lower margin.

It is evident, therefore, that determination of the relation of the ground-water body to surface flow and evaluation of pumping effects often involve geologic and hydrologic problems. Effects of pumping or diversion in upper reaches of a stream upon surface and subsurface flow in the lower stream depend upon the relation of surface flow to subflow throughout the intervening section of the stream course. It is not possible to interpret such pumping effects without investigating existing geologic and hydrologic conditions.

Conclusions

This paper has stressed some of the problems involved in court interpretations of the type of ground-water occurrence according to the present classification of subsurface water for legal purposes. Some of the erroneous legal concepts resulting from inadequate comprehension of geologic and hydrologic factors governing occurrence and movement of water underground have been discussed. In order to establish a sound classification and to formulate rulings for efficient regulation of ground-water resources, greater consideration should be given to principles of ground-water hydrology. A few general rules of law founded on well-established hydraulic engineering principles should eliminate much of the confusion that has arisen in the attempt to follow prior legal rulings based on an arbitrary classification not in agreement with occurrence of water below ground surface.

APPENDIX

Definitions

The following definitions apply to terms used in this paper.[24]

1. **Ground-Water Hydrology.** Ground-water hydrology is the branch of the science of hydrology that treats ground water; its occurrence and motions, its replenishment and depletion; the properties of rock that control ground-water movement and storage; and methods of investigation and utilization of ground water.
2. **Suspended Water.** Suspended water is water in the zone of aeration, including seeping water (gravity water) and stored water (capillary and pellicular water).

[24] Unless otherwise noted, these definitions are as given in "Ground Water," by C. F. Tolman, McGraw-Hill Book Co., Inc., 1938; see also "Outline of Ground-Water Hydrology with Definitions," by O. E. Meinzer, *U. S. Geological Survey Water-Supply Paper, No. 494*, 1923.

3. **Free Ground Water.** If water moves through interconnected openings of a body of pervious material unhampered by impervious confining material, it may be denominated "free ground water" moving under the control of the slope of the water table, as distinguished from "confined water" which moves in strata, conduits, or arteries under the control of the difference in head between the intake and discharge areas of the confined water body.
4. **Seepage.** According to generally accepted usage, seepage is the movement of water into or out of the ground. Movement of water into the ground from the surface down to the water table is called "influent seepage" and discharge of ground water to the surface is "effluent seepage." The terms "influent" and "effluent" are also used with reference to streams, ground water, and the water table. An "influent" or "losing stream" is feeding water underground and an "effluent" or "gaining stream" is receiving water from underground sources. "Influent ground water" or an "influent water table" is receiving water from the surface and "effluent ground water" or "water table" is discharging water to the ground surface.
5. **Cone of Pumping Depression.** A cone-shaped depression in the water table developed around a pumping well, the periphery of which (ground-water divide) delimits the ground water moving toward the well.
6. **Basin.** The term "basin" is popularly used for the large structural valleys of Western United States.
7. **Ground-Water Reservoir.** A "ground-water reservoir," the term commonly used for a free ground-water body, is not comparable to a static surface reservoir. Ground water is never static but is always percolating slowly from intake to discharge area in the subsurface reservoir.

   Ground water in stream-laid sand and gravel bounded by "bed and banks" of less pervious material or impervious bedrock may be considered to function as a reservoir rather than a subsurface stream if the body of gravel is bounded above and below by bedrock narrows. Like a surface reservoir the rate of flow in a ground-water reservoir is very slow, and the body of impounded water is large.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

MINIATURE SYSTEM OF FIRST-ORDER ALINEMENT AND TRIANGULATION CONTROL

By Floyd W. Hough,[1] M. Am. Soc. C. E.

Synopsis

An unusual type of survey at the Tygart Dam, on the Tygart River, 2 miles upstream from Grafton, W. Va. (Fig. 1), is described in this paper. Completed early in 1938, the structure was built by the Corps of Engineers,

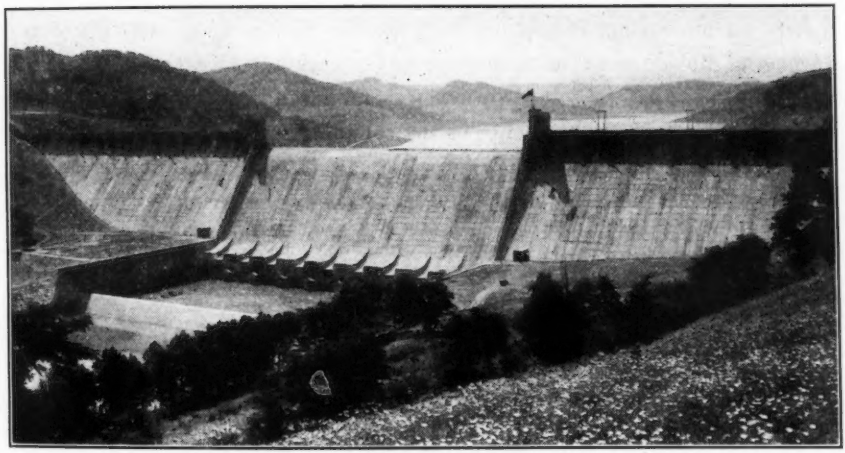

Fig. 1.—Tygart River Reservoir Dam at Grafton, W. Va.

U. S. Army (Pittsburgh (Pa.) District) and is the largest concrete gravity dam east of the Mississippi River, containing 1,250,000 cu yd of concrete. The spillway section rises 207 ft above the stream bed and has a maximum capacity of 210,000 cu ft per sec. Flanking the spillway section are non-overflow sections 23 ft higher. The total length of the dam at the top is 1,921 ft.

Tygart Dam is the first unit to be completed of the flood-control system for the protection of Pittsburgh and the upper Ohio Valley. It also serves to

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by **April 15, 1940.**

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aid navigation during the dry season on the Monongahela River (to which the Tygart River is tributary) by the release of impounded flood waters.

The method of horizontal alinement control described herein is intended not to replace, but to supplement, triangulation. As the word implies, alinement is pertinent to line rather than area control, the latter being the particular function of triangulation. Within its sphere of adaptation, the alinement method is capable of producing a greater accuracy of relative horizontal position than is the highest grade of triangulation. A survey of this type involves the study and determination of possible small horizontal movements in a fixed engineering structure, such as a large dam, as well as a study of contributing factors in the immediate vicinity. It is felt that, in such a survey, a combination of an accurate alinement survey across, and on an extension of, the structure itself, together with control of the surrounding area by first-order triangulation, furnishes a most appropriate basis for analytical study and for arrival at definite conclusions.

The purpose of the work described in this paper is to develop a suitable means for measuring, from time to time, the direction and amount of any horizontal movements that may occur in the various monoliths of the dam to supplement the data obtained from the strain gages installed in the dam, and to determine ground movements in the surrounding area. Such movements may be caused by deformation of the bed due to reservoir load, by load on the dam, temperature changes, isostatic disturbance in the vicinity of the dam, or by a combination of such forces.

The horizontal control at the Tygart Dam may be divided into two main features—alinement and first-order triangulation.

Alinement

The first-order alinement survey consisted of running straight lines across the dam, at a low stage of the reservoir pool, both longitudinally and transversely. These lines are held by small punch holes in bronze T-bars grouted flush into each monolith. The bars, ½ in. by 3½ in., at the exposed surface, were set with their long dimension at right angles to the alinement line.

The complete alinement plan and the profiles of the longitudinal lines are shown in Fig. 2. Two longitudinal lines, approximately 9 ft apart, were established across the dam, one for the alinement on the wing monoliths and the other for the spillway monoliths. The limiting clearance due to the special design of the pier houses made these two lines necessary. In addition to the alinement bars in the dam, twelve substantial concrete monuments, weighing 800 lb to 1,000 lb each, were established to hold these longitudinal alinement lines on the adjacent ridges. The main line extended to ridges 2.5 miles east

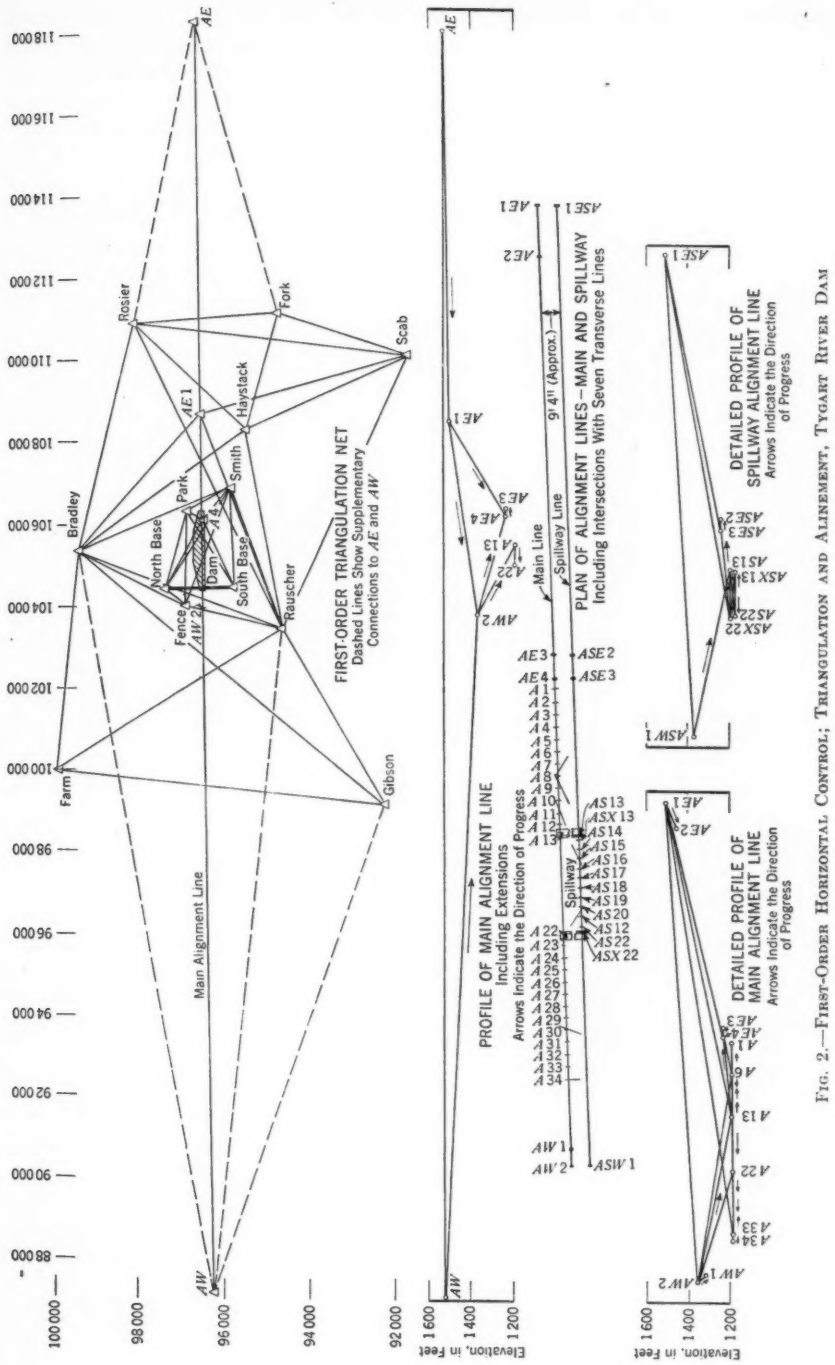

FIG. 2.—FIRST-ORDER HORIZONTAL CONTROL; TRIANGULATION AND ALINEMENT, TYGART RIVER DAM

and 3.5 miles west of the dam. Reference monuments at right angles were set except where the alinement marks were in pairs. Distances to reference marks were measured carefully by means of a light-weight standardized tape. The six transverse alinement bars were fixed in the monoliths where the various triangulation lines crossed the dam.

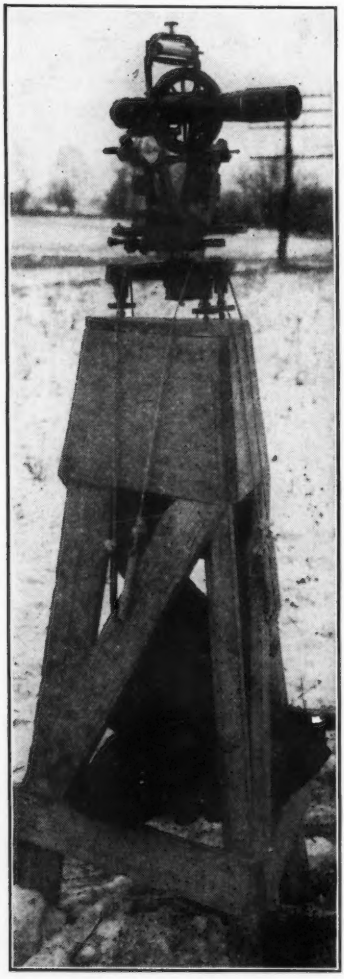

FIG. 3.—NINE-INCH DIRECTION THEODOLITE WITH MOUNTED STRIDE LEVEL

A 9-in. direction theodolite was used on this work (see Fig. 3). The telescope optics for surveys of this nature should be able to provide a high resolving power which, combined with a high magnifying power of 50, will allow the observer, in pointing, to differentiate between the center of the image and any eccentric fringes caused by atmospheric distortion of the light. The light was centered between two vertical hairs, about 35″.0 apart in the optical center of the telescope. Although a $\frac{1}{8}$-in. slit actually subtends an angle of 2″.0 at a distance of 1,000 ft, the light from this slit on an average night will occupy a width of from 4″.0 to 8″.0 in the telescope hairs, due to diffusion in the 1,000 ft of atmosphere. Experienced observers will agree that a signal can be centered between two such hairs more accurately than a single hair can be pointed upon the signal.

The observations were made entirely at night to minimize adverse atmospheric conditions. Conditions that might cause lateral refraction were avoided. The targets consisted of small electric signal lamps operating on shuffleboards. The lamps and boards were designed in the Surveying and Mapping Section of the U. S. Engineer office, at Pittsburgh, and were machined commercially to detailed specifications. Similar equipment and methods were used successfully in 1932 and 1933 by the geodetic personnel of the Metropolitan Water District of Southern California, in extending the final alinement lines for the 90 miles of tunnels of the Colorado River Aqueduct.[2]

Each signal lamp consisted of an ordinary two-cell focusing flashlight, rigidly set in a small box. The unit cells were removed and the housing rewired to allow the use of 1.5-volt ignition cells with the standard flashlight

[2] "Geodetic Engineering Applied to the Colorado River Aqueduct," by Floyd W. Hough, *Military Engineer*, March–April, 1934.

bulb. The light rays emitted from the lamp were confined to a $\frac{1}{8}$-in. vertical slit in a metal slide over the face of the lamp. At the base of the slit, and accurately centered, was a vertical hairline scratch extending down to the base of the metal frame and riding on the graduations of a section of steel tape affixed to the shuffleboard (see Fig. 4). Since, in moving the light on the shuffleboard, the slit through which all light rays passed traveled horizontally

FIG. 4.—SIGNAL LAMP AND SHUFFLEBOARD

in a vertical plane passing through the alinement station and lying at right angles to the alinement line, the effect of errors of eccentricity in the bulb or the reflector was practically eliminated. Although a narrower slit of light could have been used, it is doubtful if the accuracy of pointing the telescope would have been increased thereby. Each setting of the lamp could be read easily to the nearest 0.001 ft. No tangent screw assembly was included in the design of the board as it would have been impractical in use and would have added nothing to the accuracy of the work. The lamp was set independently at each observation by sliding it on the tracks of the shuffleboard as directed by flashlight code signals from the observer's assistant.

Where the alinement bar was less than 1,000 ft from the instrument, a small metal scale, graduated to $\frac{1}{20}$ in., was fixed by adhesive tape directly to the flat surface of the alinement bar. The target now became the point of a sharp needle scriber whose point was moved along the scale. The scale readings were then recorded for each observation, as was done with the steel tape on the shuffleboard.

The total number of observations (varying from 10 to 108) taken on any one alinement bar depended upon the range of readings and upon the distance

intervening between the instrument and the bar. The extreme range of readings obtained was closely proportional to the distance from the instrument to the bar and amounted to about 0.01 ft per 300 ft. The two master control points on the main and spillway alinement lines of monolith 13 were observed upon three and two different nights, respectively. Although these bars were 1,500 ft from the instrument, the means of the three night observations on A13 differed by a maximum of only 0.002 ft, and those for the two nights on AS13 by 0.004 ft (see Fig. 2). An abstract of the field work, listing the results of observations on different nights of the two master alinement points, is shown in Table 1.

Given experienced personnel and adequate instrumental equipment, there are many vital details involved in the methods used, the neglect of any one of which may make the difference between first-order results and those of a lower grade of accuracy. Every possible precaution was used at Tygart Dam to eliminate constant and systematic errors and to minimize accidental errors. Among these points were the following:

(*a*) Rigid lumber tripod set in ground and loaded with rocks (see Fig. 3).
(*b*) Instrument protected by observing tent.

TABLE 1.—MEAN BOARD READINGS ON DIFFERENT NIGHTS OF THE TWO MASTER ALINEMENT BARS IN MONOLITH 13

Date, 1938; April: (1)	Theodolite at station: (2)	Foresight (3)	Alinement bar* (4)	Number of observations (5)	Mean reading on board, in feet† (6)	Distance alinement bar from theodolite, in feet (7)	Punch hole south of north end, in inches (8)	Punch hole north of south end, in inches (9)	Length of bar, in inches (10)
11	AW2	AE1	A13	28	29.531	1,500	....	....	....
12	AW2	AE1	A13	40	29.529	1,500	....	....	....
13	AW2	AE1	A13	40	29.531	1,500	1.27	2.25	3.52
12	ASW1	ASE1	AS13	30	29.515	1,500	....	....	....
13	ASW1	ASE1	AS13	50	29.511	1,500	1.54	1.95	3.48

* A13 is on main alinement line in monolith 13; and AS13 is on spillway alinement line in monolith 13.
† Range, 3 nights, on A13 = 0.002 ft; and 2 nights, on AS13 = 0.004 ft.

(*c*) Accurate centering of the instrument within 0.001 ft was accomplished by using new 14-oz plumb-bobs, shielding the bob from air currents, spinning the bob to determine any eccentricity of its point, and having the instrument so oriented that the vertical plane containing the two sections of plumb-line leading from the hook coincided with the vertical plane of the alinement line.

(*d*) The theodolite was adjusted particularly for level, collimation, verticality of the pair of vertical hairs, and parallax.

(*e*) The program was so arranged that alinement points were set by observing on relatively long foresights, obviating the necessity for plunging the telescope between the pointing on the fixed light and the alinement bar to be set. This method tends to diminish any error in alinement as the instrument is moved forward to succeeding stations.

(*f*) Five observations were taken with the telescope in a direct position, followed immediately by five in the reversed position. This program of equal number of direct and reversed positions of the instrument operates to eliminate collimation errors, focusing errors of the draw tube, and errors due to the horizontal and vertical axis not being exactly at right angles to each other. At each observation, the fixed foresight light was purposely thrown out of the hairs by a small movement of the horizontal tangent screw, thus making each observation the result of an independent pointing on the fixed foresight. After setting the movable light between the hairs, the observer, before giving the "O.K." signal, habitually rested his eye by closing it momentarily and then glancing back for a check on the pointing.

(*g*) A sensitive stride level on the horizontal axis was used continuously throughout the observations instead of depending on the relatively sluggish plate level. The error due to the inclination of the vertical axis cannot be eliminated or reduced by the reversal of the instrument. An accurate stride level spanning the full length of the horizontal axis, and properly operated, is the only satisfactory answer. The stride level itself was kept in careful adjustment. Its own remaining error of adjustment was eliminated by lifting it from the horizontal axis and reversing it end for end during reversal of the instrument. The effect of this was to throw the small angle of adjustment error equally on opposite sides of the vertical plane of the alinement line. It should be noted that this took care only of the lack of perfection in the adjustment of the stride level itself. To eliminate the systematic effect of vertical axis inclination and transform it to a small compensative error, the stride level was brought to center by the instrument foot-screws after each reversal and, when necessary, immediately prior to each pointing on the fixed foresight.

(*h*) Personal error in the setting of the target on the scale was made compensative by bringing up the target alternately from the left and right. The observer at the instrument could not see the scale readings, and the person shifting the target was not in a position to see the readings on the scale until after receiving the observer's "O.K." signal.

(*i*) When using the needle scriber for a target, the illumination was alternated from equal angles to right and left of the scriber point to eliminate the error of phase.

(*j*) All readings of the target scales were checked independently by the recorder and entered in ink in the record book. The shuffleboard, which stood about 7 in. above the alinement bar, was weighted down securely. However, to furnish a positive check on the stability of the board, readings of both ends of the bar were taken on the tape both before and immediately after the observations. All computations and data in the record book were checked and initialed by both the target operator and the recorder.

(*k*) After the mean reading was determined, it was plumbed carefully the few inches from the scale on the shuffleboard down to the bronze bar and a small hole punched in the bar. In a case where the small scale was in contact with the surface of the alinement bar, the hole was punched at the mean before removing the scale. The two short distances from the punch hole to either

end of the bar and the length of the bar, for a check, were entered and initialed in the record book so that if the punch hole should become worn or even removed entirely, it may be replaced with complete assurance in its original location on the bar.

When using the aforementioned equipment and observing the precautions outlined, the alinement work can be done more accurately than even double first-order triangulation, due to the fact that neither the errors of the graduated circle nor of the micrometers are involved in the alinement observations.

From an analysis of the agreement between observations taken on different nights and the probable errors of the observations involved, it is apparent that a horizontal movement of 0.01 ft on the dam as a whole in respect to the adjacent ridges, or of half that amount of relative movement among the monoliths, can be shown conclusively by the alinement procedure described.

Triangulation

The first-order triangulation scheme consisted of sixteen stations arranged in an overlapping system of figures, concentrated mainly in the vicinity of the dam but extending east and west across valleys to adjoining ridges. The triangulation of the main scheme covered a small area of only 3.3 sq miles, with lines from 440 to 7,800 ft long, the median line being 3,450 ft. The reconnaissance was done with a particular view to proper strength of figure and avoidance of lines subject to lateral refraction.

Stations were marked by bronze plates in large concrete monuments poured in place and set 4 ft into the ground. The excavation was enlarged toward the base. This provided a batter to the monument for resisting frost action along the sides. The stations were referenced by two other concrete monuments.

The angles were observed with the direction theodolite shown in Fig. 3, with sixteen positions of the circle on each of two nights. Any directions differing by more than 4".0 from the mean for that night were rejected and a new mean taken. Observations were recorded in ink on special mimeographed forms. Computations on these record sheets were checked and initialed carefully.

The small lamps previously described, with $\frac{1}{8}$-in. vertical slits, were used as targets. The same care in centering the instrument and lamp was used on the triangulation as in the alinement work. Except for the problem of centering and the necessity for the use of an accurate stride level because of steeper vertical angles, it is scarcely more difficult to obtain good triangle closures on short lines than on long ones. However, considering that an eccentricity of 0.005 ft will cause an error of 1".0 on a line 1,000 ft long, and proportionally greater or less errors for shorter or longer lines, respectively, it is clear that the problem of centering becomes acute. That this problem may be taken care of satisfactorily is evidenced by the following statistics from the simultaneous least-squares adjustment of the first-order triangulation net at Tygart Dam:

Probable error of an observed direction	±0".47
Average correction to a direction	0".38
Average closing error of a triangle	1".07
Maximum correction to a direction	1".43
Second largest correction to a direction	1".03
Maximum closing error of a triangle	2".77
Mean error of an angle	±0".74
Average correction to an angle	0".62
Maximum correction to an angle	2".41
Number of triangles	31
Number of plus closures	18
Number of minus closures	13
Lengths of line, in feet:	
Maximum	7,800
Minimum	440
Median	3,450

There were several nights without a single rejection. The rules for rejection of observations listed in the manual of first-order triangulation[3] were rigidly followed throughout the field and office work.

The stride level was employed continuously on all observations throughout the triangulation, using the same detailed methods of operation as on the alinement. Before each half position, the stride level was centered by the instrument foot-screws with the telescope pointed approximately in the direction of the light having the greatest vertical angle. An azimuth from Polaris was observed to orient the triangulation.

A base 1,650 ft long was measured with three different 50-m invar tapes, each of which had been certified as to length, weight, and coefficient of expansion by the National Bureau of Standards. The base extended over rough ground, but the grade did not exceed 9% in any tape length. The three measurements of the base with the three different tapes fell within a range of 1.0 mm, which resulted in a computed probable error of 1 : 2,500,000.

After the lists of directions were checked and initialed, the triangulation was adjusted rigidly by the application of the method of least squares. The main scheme adjustment involved thirty-one triangles, thirty condition equations (including twenty-one angle and nine side), and sixty-eight unknowns. The relatively large number of condition equations was due to overlapping figures. The additional observed lines were introduced to strengthen the net by increasing the number of conditions to be satisfied in the adjustment. Stations AE1, AW, and AE (Fig. 2) were fitted to the main scheme by secondary adjustments.

Rectangular coordinates of all triangulation stations were computed on a horizontal plane through the crest of the dam. They were based on an assumed origin and the observed true azimuth at station FARM.

[3] *Special Publication No. 120*, U. S. Coast and Geodetic Survey.

Further elaboration as to detailed methods of triangulation, base measurement, and computations, is not made in this paper because they conformed to standardized procedure for first-order geodetic surveys already published[4] in various treatises on this subject.

Summary

The use of first-order triangulation to determine small earth movements has been employed extensively on the west coast by the U. S. Coast and Geodetic Survey. More recently the triangulation has been supplemented by alinement lines. These have been extended across known fault lines to measure small longitudinal movements.

The particular method of alinement surveys used on the Tygart Dam and for the tunnels of the Colorado River Aqueduct, as previously mentioned, has a limited field of application; but within that field it represents a most accurate means of laying a straight line on the topographical surface and of determining at a later date any changes from that line.

The "straight line" here obtained is in reality the alinement curve on the spheroid modified slightly by the deflection of the plumb-line, the latter being due to the effect of local topographic masses. This alinement curve lies between the two plane curves connecting the ends of the line and, due to the short distances between instrument stations, very nearly coincides with the geodetic line connecting the end points. In fact, for the short distances involved, all of these curves may be considered as coincident straight lines. Even in large triangulation figures, the observed direction, which is a plane curve, is treated as a geodetic line.

The deflection of the plumb-line, due to the different effects of topographic mass attraction at the various stations along the alinement line, undoubtedly does cause some deviation from the desired line; but even if it were shown that these effects were not entirely negligible from a practical standpoint, they still would not detract from the accurate determination during future surveys of movements at right angles to the alinement line. This is true because the deflection of the vertical, which is the small angle between the normals to the geoid and the spheroid at any point, may be assumed to remain constant, and the original line would be rerun under identical conditions as to topographic mass attraction, except for the changes in the reservoir mass whose effect would be negligible.

The system of overlapping triangles in a net of first-order triangulation (such as that established in the vicinity of the Tygart Dam to detect any small earth movements which might occur in the surrounding ridges) creates numerous conditions which must be fulfilled exactly to secure proper correlation between the ground points. The theory of least squares, by which the most probable values are obtained, lends itself admirably to the solution of these various condition equations.

The high order of accuracy obtained in the horizontal control at the Tygart Dam, together with the stability and permanence of the marked stations,

[4] *Special Publications Nos. 28, 120, 138*, U. S. Coast and Geodetic Survey.

furnishes an ideal condition for economical remeasurements at any future time to determine what small changes, if any, have occurred, both in amount and direction.

Although certain portions of the continent are known to be more stable (geologically speaking) than others, geologists will agree that it cannot be stated with assurance that any part of the earth's crust is entirely free from the possibility of isostatic adjustments at any unforeseen time. With this in view, does it not seem logical that 0.03% of the cost of an engineering structure, such as a large dam, may reasonably be devoted to a first-order control survey which will show conclusively just what movement, if any, is taking place in the surrounding ground as well as on the structure itself?

It is felt that, with the rapidly increasing appreciation for the economic advantages of more accurate surveys based on definite standards rigidly adhered to, such as those adopted by the International Geodetic Association and the U. S. Coast and Geodetic Survey, the time is not far distant when it will become common practice to determine the ground position of important engineering structures by the use of properly established and adjusted first-order control.

Acknowledgments

W. E. R. Covell, M. Am. Soc. C. E., Lieutenant-Colonel, Corps of Engineers, U. S. Army, is district engineer of the Pittsburgh District. The work described in this paper was done under the supervision of the writer in the Surveying and Mapping Section headed by Payson A. Perrin, Assoc. M. Am. Soc. C. E. The instrumental observations were made by John P. Mapes, junior engineer. The direction theodolite (see Fig. 3) was loaned to the District by the Director of the U. S. Coast and Geodetic Survey.

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PAPERS

PRESSURE-MOMENTUM THEORY APPLIED TO THE BROAD-CRESTED WEIR

By H. A. Doeringsfeld,[1] Esq., and C. L. Barker,[2] Assoc. M. Am. Soc. C. E.

Synopsis

The general theory of pressure momentum, and its application to the broad-crested weir, is presented in this paper, with data obtained by test. The purpose of the experimental work was to check a formula for flow over the weir developed on the basis of conservation of momentum. The application also applies to the sharp-edged entrance to flumes from reservoirs.

Introduction

The advantages of the broad-crested weir as a measuring device, where head is at a premium, have been enumerated by several experimenters and most recently by James G. Woodburn,[3] M. Am. Soc. C. E. The pressure-momentum theory offers a method of developing a formula for the flow of water which is free from empirical coefficients that vary widely with different heads and crest heights. The use of the formulas requires only the measurement of the head on the weir.

Notation.—The symbols used in this paper are defined where they first appear and are assembled for reference in the Appendix.

Review of the Pressure-Momentum Theory

Consider the part of a filament of water, $\Delta l'$ in length, whose cross-sectional areas at points 1 and 2 are ΔA_1 and ΔA_2, respectively (see Fig. 1). Let the pressures at points 1 and 2 be P_1 and P_2, respectively. If the inclination of the filament to the horizontal is θ, the resultant force in the direction of motion (that is, downward and to the right) will be the algebraic sum of the gravity and pressure components in that direction. Equating the total force to the

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by April 15, 1940.

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[3] "Tests of Broad-Crested Weirs," by James G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 387.

product of mass and acceleration (which is the rate of change of momentum):

$$P_1 \Delta A_1 - P_2 \Delta A_2 + \Delta G \sin \theta = \Delta m \frac{dV}{dt} \quad \ldots\ldots\ldots\ldots (1)$$

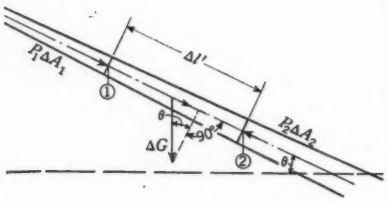

FIG. 1

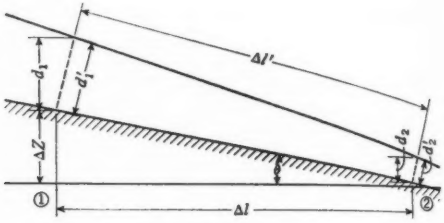

FIG. 2

in which m is the mass; G is the gravatational force; V is the velocity; and t is time. Since the volume of the segment is nearly $\frac{(\Delta A_1 + \Delta A_2)\, \Delta l'}{2}$,

$$\Delta G \sin \theta = \tfrac{1}{2} (\Delta A_1 + \Delta A_2)\, \Delta l'\, w \sin \theta \quad \ldots\ldots\ldots\ldots (2)$$

in which w is the weight of water per unit volume. Substituting Equation (2) in Equation (1), multiplying by dt, and integrating:

$$(P_1 \Delta A_1 - P_2 \Delta A_2 + \Delta G \sin \theta)\, t = \Delta m\, V_2' - \Delta m\, V_1' \quad \ldots\ldots\ldots (3)$$

in which V_1' and V_2' are the velocities at points 1 and 2, respectively, in the direction $\Delta l'$. Substituting $\Delta m = \frac{\Delta Q\, w\, t}{g}$, dividing by t, and transposing:

$$\left(P_1 \Delta A_1 + \frac{\Delta Q\, w\, V_1'}{g} \right) - \left(P_2 \Delta A_2 + \frac{\Delta Q\, w\, V_2'}{g} \right) + \tfrac{1}{2} (\Delta A_1 + \Delta A_2)\, \Delta l'\, w \sin \theta = 0 \quad \ldots\ldots\ldots\ldots (4)$$

in which g = acceleration due to gravity.

Consider now an open channel of length Δl, depth d, width b, and inclination θ (Fig. 2). From Equation (4), assuming parallel filaments of flow,

$$\sum_0^{d_1'} \left(P_1 \Delta A_1' + \frac{\Delta Q\, w\, V_1'}{g} \right) - \sum_0^{d_2'} \left(P_2 \Delta A_2' + \frac{\Delta Q\, w\, V_2'}{g} \right) + \sum_0^{0.5(d_1' + d_2')} \frac{(\Delta A_1' + \Delta A_2')}{2} \Delta l'\, w \sin \theta = 0 \quad \ldots\ldots\ldots\ldots (5)$$

but

$$dA_1' = b\, \mathrm{d}d_1' \quad \ldots\ldots\ldots\ldots (6a)$$

$$dA_2' = b\, \mathrm{d}d_2' \quad \ldots\ldots\ldots\ldots (6b)$$

$$dQ = b\, \mathrm{d}d_1'\, V_1' = b\, \mathrm{d}d_2'\, V_2' \quad \ldots\ldots\ldots\ldots (6c)$$

in which b = the breadth of the channel. In Equation (5), replacing the Δ's by differentials and substituting the values in Equations (6):

$$\int_0^{d_1} b\left[P_1\,dd_1' + \frac{dd_1'\,(V_1')^2}{g}w\right] - \int_0^{d_2'} b\left[P_2\,dd_2' + \frac{dd_2'\,(V_2')^2}{g}w\right] + \frac{b\,(d_1' + d_2')}{2}\Delta l'\,w\sin\theta = 0 \quad \ldots\ldots(7)$$

Furthermore, since $P = w\,d$; $d = \frac{d'}{\cos\theta}$; $V' = \frac{V}{\cos\theta}$; and $\Delta l = \frac{\Delta l'}{\cos\theta}$:

$$P = \frac{w\,d'}{\cos\theta} \quad \ldots\ldots(8)$$

Integrating Equation (7) and substituting the foregoing values:

$$b\left(\frac{d_1^2}{2} + \frac{V_1^2\,d_1}{g\cos^2\theta}\right)w - b\left(\frac{d_2^2}{2} + \frac{V_2^2\,d_2}{g\cos^2\theta}\right)w + b\,\frac{(d_1 + d_2)}{2}\Delta l\,w\tan\theta = 0 \quad \ldots\ldots(9)$$

If $\theta = 0$ and $\frac{V^2}{g} = 2H$, the pressure-momentum equation for a horizontal channel becomes:

$$b\,w\left[\left(\frac{d_1^2}{2} + 2H_1\,d_1\right) - \left(\frac{d_2^2}{2} + 2H_2\,d_2\right)\right] = 0 \quad \ldots\ldots(10)$$

in which H = the hydraulic head. If friction forces are considered, the right-hand member of Equation (10) is not zero, but is set equal to the sum of these forces, f. Hence, if

$$F = b\,w\left(\frac{d^2}{2} + 2\,dH\right) \quad \ldots\ldots(11)$$

then

$$F_1 - F_2 - f = 0 \quad \ldots\ldots(12)$$

in which F is a pressure-plus-momentum force.

Application of Theory to Broad-Crested Weir

Consider the broad-crested weir sketched in Fig. 3. The pressure-plus-momentum force in Equation (8), on the upstream side, is:

$$F_1 = b\,w\left(\frac{d_1^2}{2} + \frac{Q^2}{g\,b^2\,d_1}\right) \quad \ldots\ldots(13a)$$

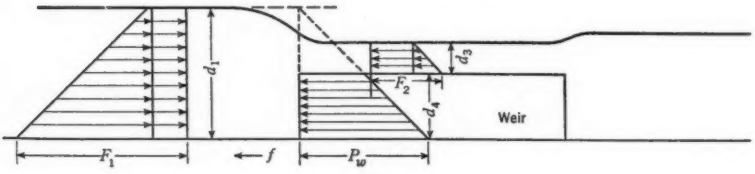

Fig. 3

On the top of the weir:

$$F_2 = b\,w\left(\frac{d_3^2}{2} + \frac{Q^2}{g\,b^2\,d_3}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (13b)$$

The force P_w in Fig. 3, due to unit water pressure p on the weir face, is assumed to be:

$$P_w = w\,b\,p\left[\frac{d_1 + (d_1 - d_4)}{2}\right] = w\,b\,d_4\left(\frac{2\,d_1 - d_4}{2}\right) \quad \ldots\ldots\ldots (14)$$

The accuracy of this assumption is checked experimentally. Substituting Equations (13) and (14) in Equation (12), and simplifying:

$$\left(\frac{d_1^2}{2} + \frac{Q}{g\,b^2\,d_1}\right) - \left(\frac{d_3^2}{2} + \frac{Q^2}{g\,d^2\,b_3}\right) - \left(\frac{2\,d_1 - d_4}{2}\right) d_4 = 0 \quad \ldots\ldots (15)$$

In Equation (15), let $d_1 - d_4 = H$; and $H = K\,d_3$, in which K = a constant. Then simplifying and solving for Q:

$$Q = \frac{b\sqrt{2\,g}}{2}\,H^{1.5}\left\{\left(\frac{K^2 - 1}{K^2}\right)\left[\frac{d_4 + H}{K\,(d_4 + H) - H}\right]\right\}^{0.5} \quad \ldots\ldots\ldots (16)$$

In order to reduce Equation (16) to the form commonly developed for weirs, it may be written:

$$Q = \Omega\,b\,H^{1.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which

$$\Omega = \frac{\sqrt{2\,g}}{2}\left\{\left(\frac{K^2 - 1}{K^2}\right)\left[\frac{d_4 + H}{K\,(d_4 + H) - H}\right]\right\}^{0.5} \quad \ldots\ldots\ldots\ldots (18)$$

Assuming the flow across the weir to be parallel, the pressure distribution on the upstream face to be as assumed, and the frictional losses between the crest and the position of parallel flow to be negligible, Equation (16) should give the discharge. To check these assumptions and to evaluate K the following apparatus was developed.

Description of the Apparatus and Test Procedure

Minnesota Tests.—A metal weir was constructed of two standard 6-in. channels and one $\frac{1}{4}$-in. steel plate 1.68 ft by 2.25 ft. The channels that formed the upstream and downstream faces were screwed to the plate that formed the top of the weir. The resulting crest height of the completed weir was 0.53 ft; the breadth of the weir was the width of the channel, 1.68 ft; and the length was 2.25 ft. Holes for piezometer connections were drilled on the center lines of both ends and the top. On the faces, the top hole was drilled as near the edge as possible; the first five holes were spaced 0.025 ft on centers and the remaining holes, 0.05 ft on centers. On the crest, holes were drilled as near the edge as possible; the first five holes from the edge were spaced 0.05 ft on centers and the remaining holes were spaced 0.1 ft on centers. Into each hole drilled in the weir was screwed a brass nipple carrying a hole $\frac{1}{16}$ in. in diameter. The ends of the nipples terminating on the surface of the weir were made very smooth so that there would be no disturbance due to them.

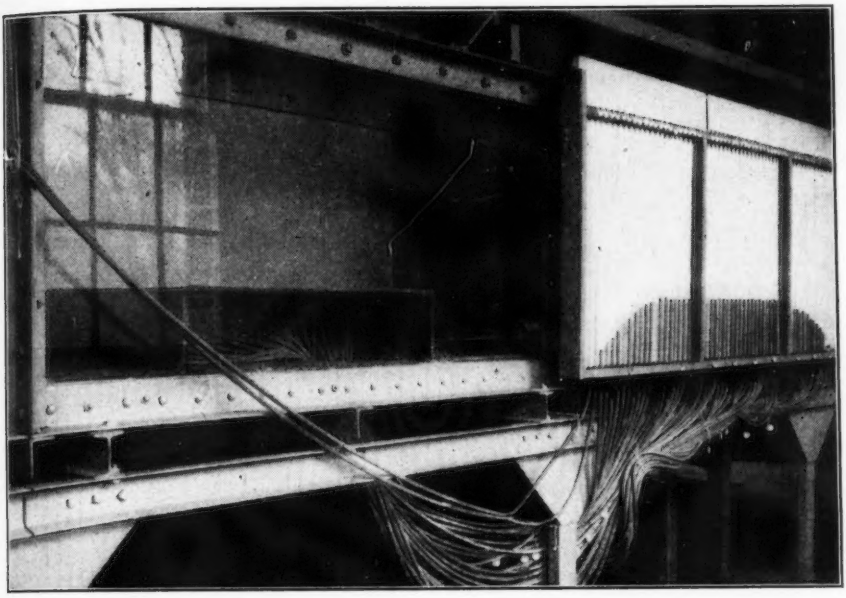

FIG. 4.—THE METAL BROAD-CRESTED WEIR AND PIEZOMETER TUBES

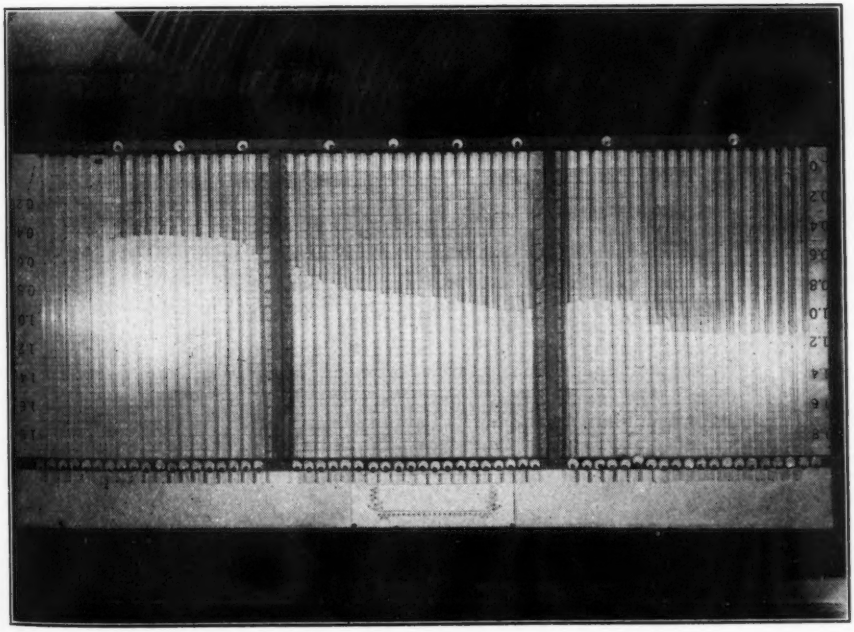

FIG. 5.—THE BANK OF PIEZOMETER TUBES IN OPERATION SHOWING THE PRESSURE ON THE UPSTREAM FACE AND TOP OF THE WEIR

The rubber tubes led from the weir nipples to a rack containing sixty glass piezometer tubes $\frac{1}{4}$ in. in diameter (see Figs. 4 and 5). Two of the glass tubes were connected to read the head on the weir 15 ft upstream. Two more were connected to read the head several feet downstream from the weir.

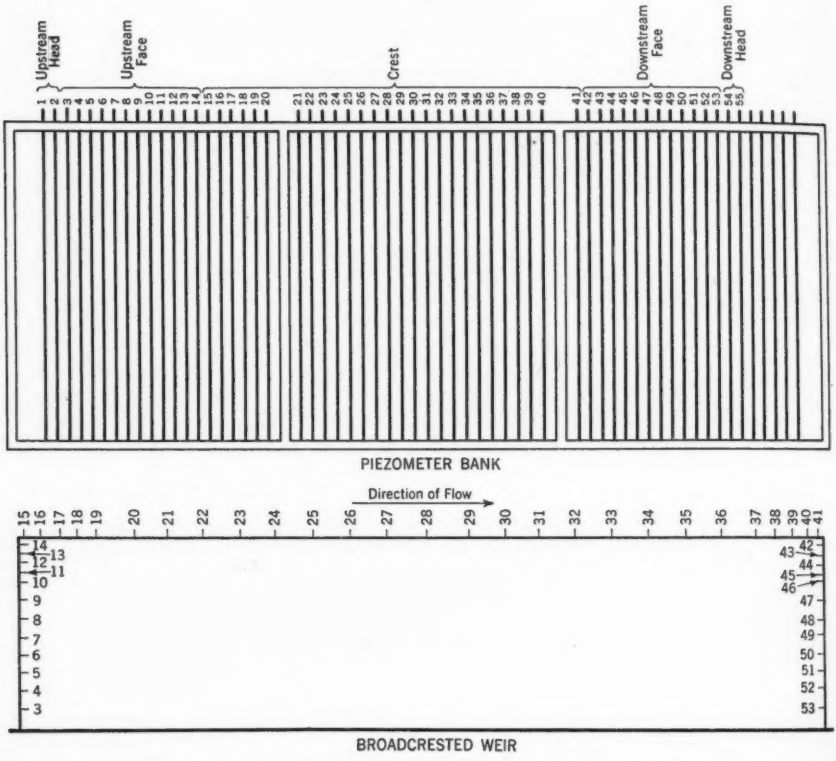

FIG. 6.—BROAD-CRESTED WEIR SHOWING NIPPLES AND NUMBERS; NOS. 1 AND 2 CONNECTED UPSTREAM AND NOS. 52 AND 53 CONNECTED DOWNSTREAM

Fig. 6 shows the location of the nipples on the weir and the numbers of the nipples. These nipples were connected by the rubber tubes mentioned to the correspondingly numbered piezometer tubes in the rock. Piezometer tubes 1 and 2 were connected upstream to measure the head. Nipples 3 to 14 inclusive were on the upstream side.

A scale reading to 0.01 ft was placed behind the piezometer tubes. With the apparatus thus constructed it was possible to read the pressure profile over the weir at a glance. The water-surface profile was taken with a point gage mounted on a track on top of the glass channel (see Figs. 4 and 7). In Fig. 7 the sharp-crested weir is in the foreground. This leads to the glass channel and on to the broad-crested weir. The rails carrying the profile gage are shown on top of the channel.

FIG. 7.—VIEW OF LABORATORY SET-UP

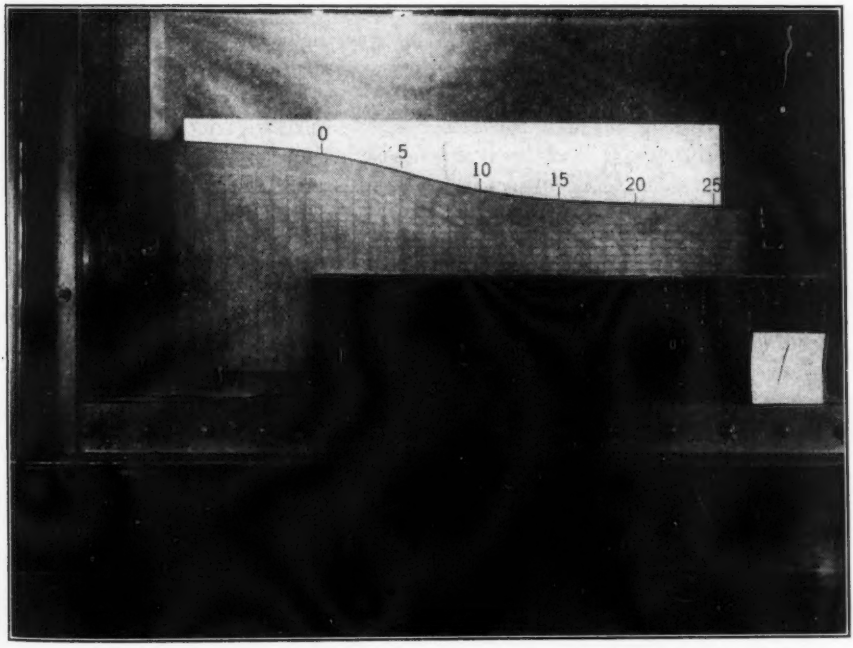

FIG. 8.—DIRECTION OF THE FLOW LINES AS INDICATED BY THE THREADS IN NEEDLES

To trace out the flow lines, 2-in. strands of No. 50 black silk thread tied in the eyes of No. 9 needles, which were mounted perpendicular to a white board in columns 1 in. apart, were used. The needles were spaced vertically $\frac{1}{4}$ to $\frac{1}{2}$ in. Photographs were taken of the threads trailing in the stream to provide a permanent record (see Fig. 8).

The water used in the experiments was circulated by pumps in the basement of the laboratory. These forced the water to an overhead constant-level tank out of which the water was carried through a 10-in. header to a sharp-crested rectangular weir (see Fig. 7). The head on the weir was measured by a hook gage. The discharge from the weir was carried by a glass channel 2.5 ft deep, 1.68 ft wide, and 40 ft long. The broad-crested weir under test was placed just downstream from the middle of the channel. The lower end of the glass channel was closed by a gate that could be adjusted to regulate the tailwater elevation on the broad-crested weir. The channel discharged into two 11,000-lb weighing tanks. Before any experimental work was performed, the tanks were calibrated and then used to calibrate the sharp-crested weir at the entrance of the channel. This calibration was checked frequently during the experiment to obviate any errors.

Eleven runs were made under heads varying from about 0.1 ft to 0.6 ft, for each of which the water-surface profile was taken with the point gage. On these runs pressures were read on the weir by means of the piezometers described.

Five weirs of different crest heights, and the same crest length and breadth as the steel weir just described, were placed in the glass channel. The data are shown graphically in Fig. 9.

Washington Tests.—Two weirs were constructed of tempered hard board, with crest heights of 0.262 and 0.471 ft, crest lengths of 0.833 and 0.833 ft, and crest breadths of 1.12 and 1.65 ft, respectively. These weirs were placed in a channel so arranged that the water-surface profile could be determined. The water flowing through the channel was measured by a sharp-crested, constricted weir whose head was measured to 0.001 ft. The head on the broad-crested weir was decreased in nearly uniform increments until standing waves formed on the surface of the weir.

Results

Fig. 10 shows the results obtained as to water depth and pressure distribution over the weir. They check, substantially, the results observed by Professor Woodburn with regard to the limiting depth of tailwater without affecting the head on the weir; and the position of critical depth (d_c) agrees quite well with the locations obtained in his experiments. However, the critical depth was computed on the basis of parallel flow by the formula

$$d_c = \frac{\sqrt[3]{q^2}}{g} \qquad (19)$$

which cannot be assumed to be the critical depth for curvilinear flow, the condition that exists at the upstream edge of the weir.[4] In Equation (19) q = flow through a unit of cross section of area.

[4] "Hydraulics of Open Channels," by Boris A. Bakhmeteff, M. Am. Soc. C. E., p. 43.

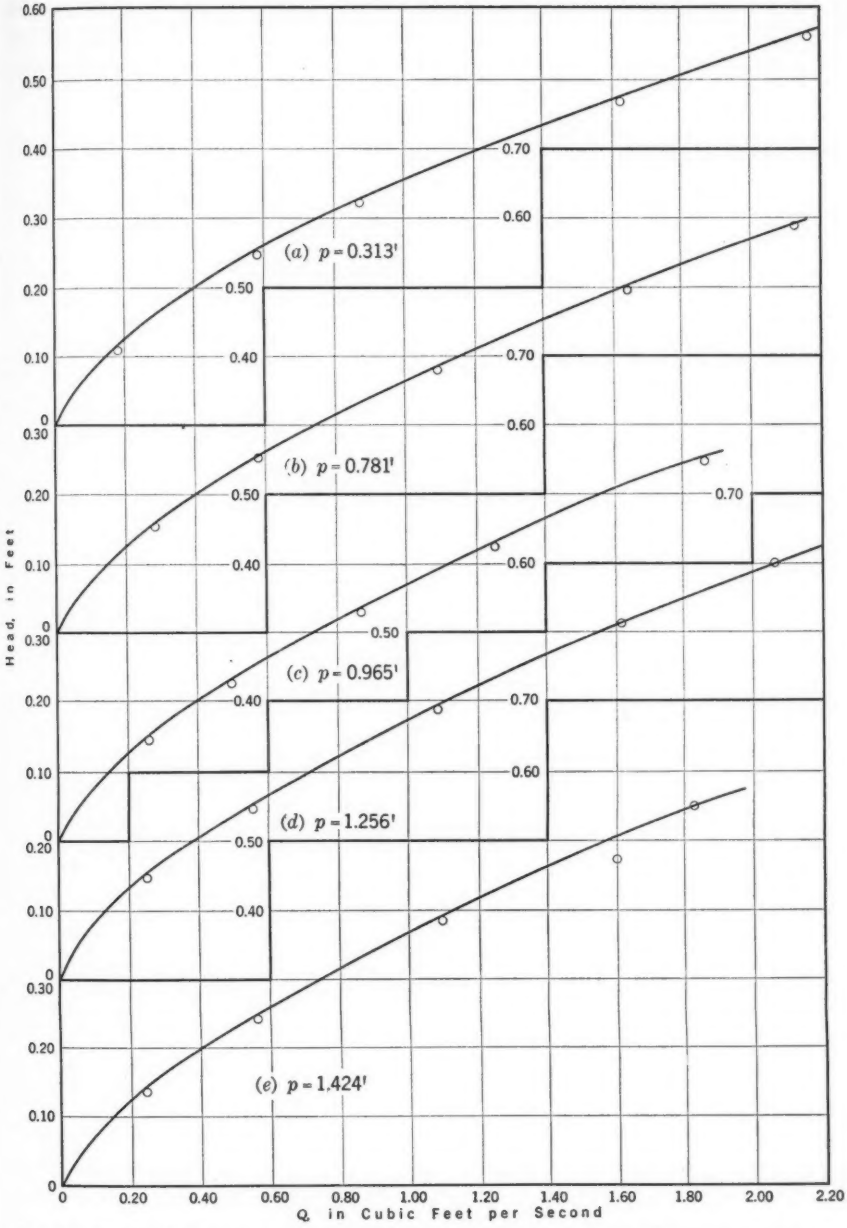

FIG. 9.—CURVES COMPUTED FROM EQUATION (16) AND THE CORRESPONDING EXPERIMENTAL DATA

An examination of the pressure profiles substantiates the assumption made in the development of Equation (15) that the pressure on the upstream face was due to the depth d_1. The study of the water-surface profiles and the lines of flow taken from the photographs gave the positions of d_3 (depth of parallel

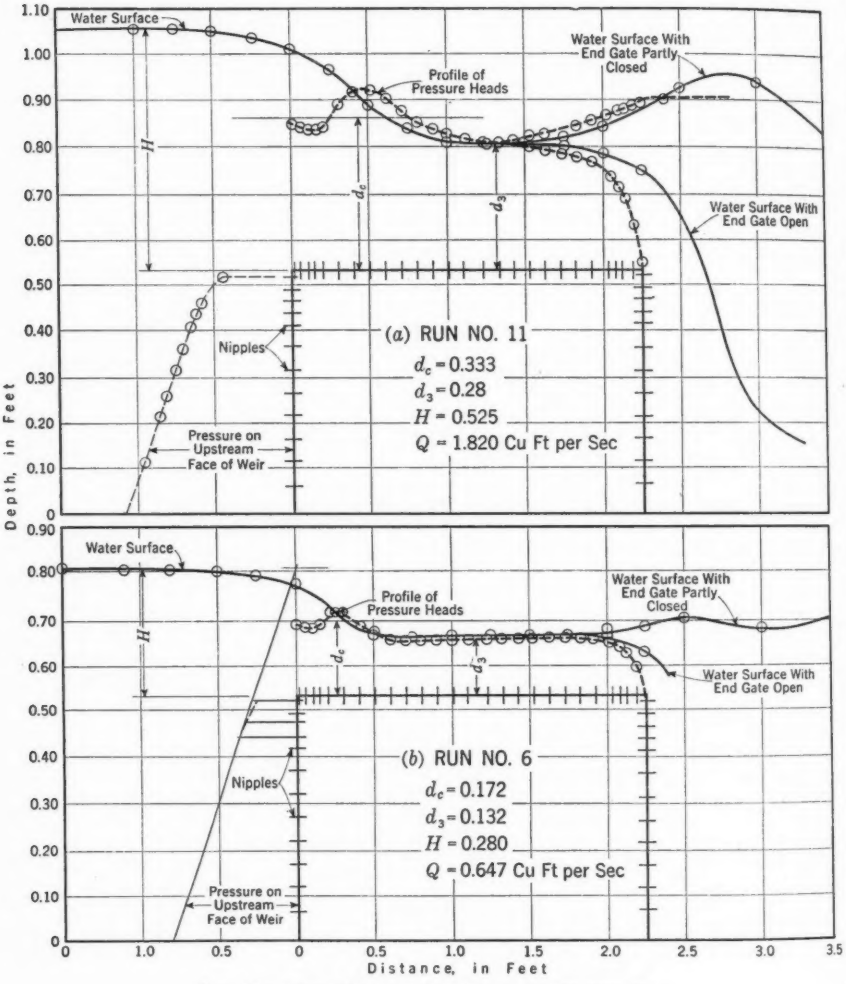
Water Surface
Water Surface With End Gate Partly Closed
Profile of Pressure Heads
Water Surface With End Gate Open
Nipples
Pressure on Upstream Face of Weir
(a) RUN NO. 11
d_c = 0.333
d_3 = 0.28
H = 0.525
Q = 1.820 Cu Ft per Sec
(b) RUN NO. 6
d_c = 0.172
d_3 = 0.132
H = 0.280
Q = 0.647 Cu Ft per Sec
Depth, in Feet
Distance, in Feet

FIG. 10.—WATER DEPTHS AND PRESSURE DISTRIBUTION

flow across the weir) for the different runs. At the point of minimum depth, for a very short stretch, the water is flowing parallel (or nearly so) to the surface of the weir. It was at this point that d_3 was measured.

The values of K on the Minnesota weir are given in Table 1(a), and Table 1(b) shows the values of K determined at Washington State College. Although there is some variation in the average values of K determined, the formula is

rather insensitive to the value of K. The average of all the various averages is 1.98. If a value of 2 is used for K, to make computations simple, the results given by Equation (16) are quite close for practical computations.

TABLE 1.—OBSERVED VALUES OF K

(a) MINNESOTA TESTS			(b) WASHINGTON TESTS					
Crest Dimensions, in Feet: Height 0.53, Length* 1.68, Breadth 2.25			Height 0.262, Length* 0.833, Breadth 1.12				Height 0.471, Length* 0.833, Breadth 1.65	
Head, H, in feet	Depth, d_3, in feet	$K = \frac{H}{d_3}$	Head, in feet	K	Head, in feet	K	Head, in feet	K
(1)	(2)	(3)	(1)	(3)	(1)	(3)	(1)	(3)
0.117	0.060	1.95	0.053†	1.29	0.207	1.83	0.076	1.73
0.128	0.070	1.83	0.090‡	1.49	0.226	1.86	0.088	1.91
0.156	0.085	1.84	0.103‡	1.64	0.228	1.90	0.108	1.96
0.230	0.120	1.92	0.124‡	1.70	0.245	1.90	0.121	1.92
0.270	0.140	1.93	0.132	1.86	0.268	1.88	0.144	1.95
0.355	0.180	1.97	0.151	1.92	0.279	1.86	0.159	2.09
0.430	0.220	1.96	0.157	1.84	0.295	1.88	0.180	2.12
0.480	0.250	1.92	0.170	1.77	0.302	1.90	0.194	2.11
0.528	0.280	1.88	0.186	1.86	0.334	1.90	0.217	2.04
0.585	0.310	1.88	0.195	1.82	....	....	0.232	2.08
0.552	0.285	1.94					0.251	2.04
							0.268	2.04
							0.288	2.01
Average	....	1.91	....	....	....	1.865§	....	2.06

* Channel width. † Ripples on crest.
‡ Standing waves on weir. § Average of last fifteen values.

ACCURACY OF THE FORMULA

Twenty-six tests were made on the steel weir at Minnesota, including the eleven in Table 1(*a*), to check the accuracy of Equation (16) (see Table 2). The discharge by the sharp-crested weir is taken as correct. The discharge for the broad-crested weir is determined by Equation (16). In the first case (Column (3), Table 2) K is used as 1.93 which was the average for the eleven runs on this weir. In the second case (Column (4), Table 2) K is used as 2.00 to show the effect of a slight change upon the formula.

From Table 2, for the steel weir used in the test, the value of K could be well assumed as 2. Column (9), Table 2, shows the quantities as computed by Equation (10), using K as 2 for the various weirs. Data published by the U. S. Geological Survey[5] were used for check computations, and results are given in Table 3.

CONCLUSION

It is the writers' opinion, based on the tests and computations given, that flow over a broad-crested weir may be computed quite accurately by use of Equations (16) and (17), assuming K to be 2. Thus far tests were made on only sharp-edged weirs. Undoubtedly a similar expression could be developed for round-edged weirs. The accuracy of the formulas is to be questioned if the breadth of the weir crest is so small that the water surface does not become

[5] *Water Supply Paper No. 200*, U. S. Geological Survey, p. 70.

TABLE 2.—Tests to Check the Accuracy of Equation (16)

Head on broad-crested weir, in feet; crest height d_4 = 0.53	Discharge by Sharp-Crested Weir, in Cubic Feet per Second			Percentage Error, by Equation (16)		Head on broad-crested weir, in feet; crest height d_4 = 0.262	Discharge by Sharp-Crested Weir, in Cubic Feet per Second			Percentage Error, by Equation (16)	
	Observed	By Equation (16)		K = 1.93	K = 2.00		Observed	By Equation (16) (b = 0.833; and d_4 = 0.262)*		K = 2.00	K = 1.865‡
		K = 1.93	K = 2.00					K = 2.00	K = 1.865‡		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0.068	0.075	0.0760	0.0750	+1.30	0.00	0.053*	0.0270	0.0262	0.0264	− 2.86	−2.22
0.112	0.160	0.1612	0.1617	+0.60	+1.06	0.090*	0.0650	0.0592	0.0599	− 9.50	−7.90
0.114	0.165	0.1670	0.1660	+1.20	+0.73	0.103*	0.0790	0.0732	0.0740	−13.00	−6.33
0.117	0.170	0.1745	0.1732	+2.60	+1.76	0.124*	0.1000	0.0977	0.0993	− 2.30	−0.70
0.128	0.193	0.2000	0.1983	+3.50	+2.62	0.132	0.1100	0.1078	0.1093	− 2.00	−0.64
0.144	0.240	0.2410	0.2380	+0.40	−0.83	0.151	0.1348	0.1331	0.1355	− 1.27	+0.52
0.155	0.270	0.2690	0.2667	−0.37	−0.11	0.157	0.1420	0.1415	0.1435	− 0.35	+1.05
0.156	0.270	0.2720	0.2693	+0.74	−0.26	0.170	0.1650	0.1602	0.1626	− 2.90	−1.56
0.178	0.330	0.3340	0.3300	+1.20	0.00	0.186	0.1810	0.1844	0.1886	+ 1.88	+4.20
0.230	0.500	0.4950	0.4940	−1.00	−1.30	0.195	0.2040	0.1990	0.2021	− 2.45	−0.93
0.270	0.647	0.6410	0.6340	−0.90	−2.00	0.207	0.2270	0.2189	0.2222	− 3.96	−2.12
0.289	0.700	0.7120	0.7080	+1.70	+0.87	0.226	0.2620	0.2513	0.2557	− 4.08	−2.40
0.334	0.890	0.8940	0.8860	+0.45	−0.45	0.228	0.2615	0.2550	0.2605	− 2.49	−0.39
0.355	0.955	0.9830	0.9730	+3.00	+2.30	0.245	0.2910	0.2853	0.2907	− 1.95	−0.10
0.390	1.140	1.1470	1.1300	+0.60	−0.88	0.268	0.3350	0.3290	0.3354	− 1.79	+0.12
0.430	1.280	1.3330	1.3180	+4.10	−3.14	0.279	0.3500	0.3509	0.3575	+ 0.26	+0.21
0.434	1.290	1.3500	1.3380	+4.70	+3.72	0.295	0.3850	0.3830	0.3908	− 0.52	+1.49
0.452	1.420	1.4430	1.4250	+1.60	+0.35	0.302	0.4180	0.3972	0.4053	− 5.02	−3.11
0.480	1.560	1.5580	1.5670	−1.80	+0.45	0.334	0.4550	0.4669	0.4765	+ 2.53	+4.73
0.481	1.580	1.5900	1.5720	+0.60	−0.51	0.108*	0.0770	0.0768†	0.0752†	− 0.26	−2.24
0.508	1.730	1.7400	1.7200	0.60	0.58	0.121*	0.0940	0.0915†	0.0900†	− 2.66	−4.25
0.528	1.820	1.8500	1.8230	+1.60	+1.65	0.144*	0.1200	0.1197†	0.1176†	− 0.25	−2.00
0.552	1.973	1.9800	1.9550	+0.35	−0.91	0.159	0.1440	0.1395†	0.1373†	− 3.12	−4.65
0.555	1.975	1.9950	2.0770	+1.01	+0.52	0.180	0.1680	0.1692†	0.1663†	+ 0.72	−1.01
0.587	2.130	2.1800	2.1570	+2.35	+1.27	0.194	0.1820	0.1898†	0.1871†	+ 4.10	+2.80
0.588	2.150	2.1900	2.1620	+1.40	+0.56	0.217	0.2300	0.2260†	0.2225†	− 1.74	−3.26
						0.235	0.2530	0.2560†	0.2521†	+ 1.19	−0.36
						0.251	0.2830	0.2837†	0.2798†	+ 0.25	−1.13
						0.268	0.3020	0.3140†	0.3097†	+ 4.00	+2.55
						0.288	0.3470	0.3515†	0.3468†	+ 1.30	−0.05

* Ripples and standing waves on crest.
† In the last eleven tests d_4 = 0.471.
‡ In the last eleven tests K = 2.08.

TABLE 3.—Experiments on Weirs of Irregular Section

(a) Bazin's Series No. 115: Crest Height = 2.46 Ft; and Crest Length = 6.56 Ft			(b) Cornell Experiments:* Crest Height = 11.25 Ft; and Crest Length = 15.969 Ft		
Observed head, in feet	Coefficient $C_1 = \frac{Q}{L H^{1.5}}$	Value of Ω by Equation (17)	Observed head, in feet	Coefficient $C_1 = \frac{Q}{L H^{1.5}}$	Value of Ω by Equation (17)
(1)	(2)	(3)	(1)	(2)	(3)
0.196	2.25	2.50	1.023	2.61	2.65	0.312	2.684	2.47
0.264	2.41	2.52	1.097	2.63	2.67	0.447	2.713	2.48
0.342	2.45	2.53	1.178	2.64	2.68	0.698	2.696	2.49
0.415	2.51	2.55	1.260	2.65	2.69	0.851	2.579	2.50
0.495	2.50	2.57	1.330	2.68	2.70	1.206	2.652	2.52
0.566	2.55	2.58	1.388	2.69	2.71	1.480	2.531	2.53
0.638	2.54	2.59	1.424	2.70	2.71	1.890	2.624	2.54
0.716	2.56	2.61	1.467	2.70	2.72	2.360	2.637	2.57
0.792	2.60	2.62				2.935	2.608	2.60
0.871	2.60	2.63				3.661	2.637	2.62
0.948	2.60	2.64				4.436	2.647	2.65

* *Water Supply Paper No. 200*, U. S. Geological Survey, Series XLIII, p. 70.

parallel to the surface of the weir, or nearly so. Raising the surface of the tailwater had no effect on any runs until the point of parallel flow was shifted from the position it had with the tailwater down. Hence, the formulas can be used where some of the head is being regained by raising the tailwater.

Acknowledgment

The experiments upon which this paper is based were originally started at the Experimental Engineering Laboratory, University of Minnesota, Minneapolis, Minn., under the direction of Lorenz G. Straub, Assoc. M. Am. Soc. C. E. Later experiments were made in the Hydraulic Laboratory at the State College of Washington, Pullman, Wash. Other sources of information are cited by footnote references in the text.

APPENDIX

The following notation conforms essentially with American Standard Symbols for Hydraulics,[6] compiled by a committee of the American Standards Association with Society representation, and approved by the Association in 1929.

A = area of cross section; ΔA = increment of area;
b = breadth;
C = coefficient; $C_1 = L\,H^{1.5}$;
d = depth of flow; d' = depth perpendicular to the bottom slope; d_c = critical depth; and d_1, d_2, and d_3 are as shown in Fig. 3;
F = pressure-plus-momentum force;
f = sum of all friction forces;
G = gravitational force; ΔG = increment force;
g = acceleration due to gravity;
H = hydraulic head;
K = a constant $= \frac{H}{d}$;
L = length;
l = length of a filament of water; Δl = increment of length;
m = mass;
P = total pressure; P_1 and P_2 = total hydraulic pressure on the cross sections at points 1 and 2, respectively; P_w = force due to water pressure on the weir face;
p = unit pressure intensity;
Q = rate of flow;
q = rate of flow through a unit cross section;
t = time;
V = velocity; V' = velocity of a given point, in the direction of $\Delta l'$;
w = weight of water per unit volume;
θ = inclination of a water filament with the horizontal; also, θ = slope of the bottom of the channel.

[6] A. S. A.—Z10b—1929.

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PAPERS

NORRIS DAM CONSTRUCTION CABLEWAYS

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Synopsis

The development and application of thrust-wheel type traveling cableways and the detailed features of design at Norris Dam are here explained. The design features were checked under field conditions by tests which are described, analyzed, and compared to the design figures. In general the tests check the original computations closely, but in some instances they pointed to opportunities for further study and precautions to be used in the application of theoretical formulas.

Preliminary Investigation

Norris Dam was the third construction project to use heavy duty cableways supported on traveling towers specially constructed to take the horizontal thrust entirely through a set of horizontal thrust wheels. The first was the Madden Dam in Panama, with a span of 1,325 ft. Soon after that successful use, three similar installations were made at Hoover Dam, with spans ranging from 1,365 ft to 2,575 ft.

Previous to these installations, when traveling type cableways were used the towers were usually mounted on the heaviest kind of railroad trucks, and the entire horizontal thrust was carried on the wheel flanges. As greater projects were undertaken, loads were increased and larger sized concrete buckets were used, requiring an increased number of vertical wheels to support the towers and to carry the horizontal thrust. The excessive flange loads caused a great many broken axles, and shutdowns while repairs were being made, with additional expense. The numerous axle failures on the Owyhe Dam cableway were typical of this experience.

For the Norris Dam operations 6-cu yd concrete buckets were contemplated. With a cableway span of 1,925.5 ft, this load would cause a horizontal thrust on the towers of more than 200 tons, which is too great to carry on wheel flanges. A thrust-wheel type tower was clearly indicated under these conditions. To handle concrete in smaller buckets would have required a slower construction

Note.—Written comments are invited for immediate publication; to ensure publication the last discussion should be submitted by April 15, 1940.

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schedule, and the cost of placing the concrete would have been correspondingly higher. The two Norris Dam cableways together placed 1,005,000 cu yd of concrete and handled more than 68,000 tons of other materials during the construction of the dam.

The topography on each side of the river was such that runways could be built economically. The right bank of the river facing downstream was a rocky bluff, sloping about 45° with a suitable location for the head-tower runway about 105 ft above the roadway on top of the dam. The center part of this runway was a rocky shelf which was excavated from the bluff and which sloped off at the extremities, necessitating the use of a heavy steel trestle at each end of the runway. The tail-tower runway on the left bank was constructed entirely on earth fill. The topography permitted this fill to be built to the same elevation as the roadway on top of the dam, and later to be used as part of the permanent access road.

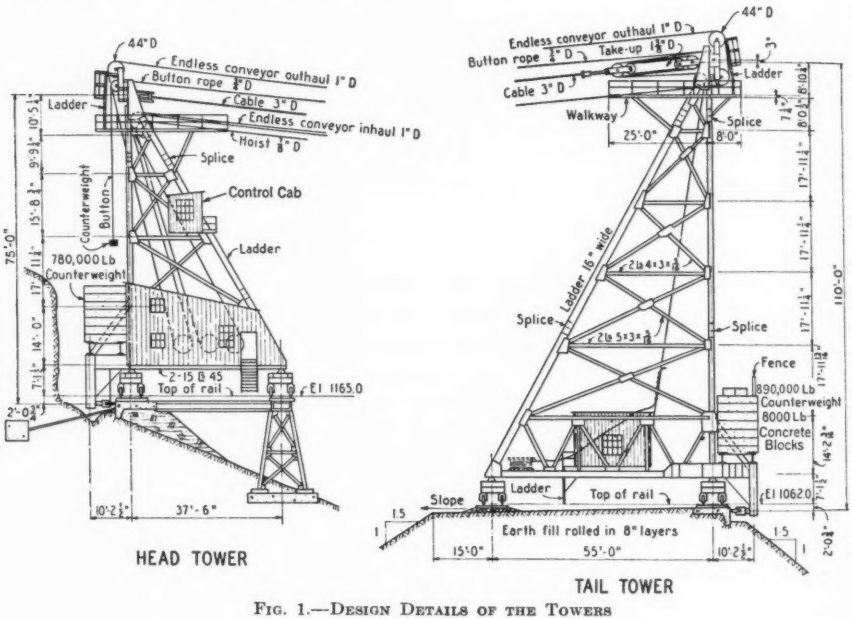


FIG. 1.—DESIGN DETAILS OF THE TOWERS

The thrust-wheel type cableway (Fig. 1) permits the entire horizontal thrust to be placed on the back side of the runway; whereas, on the vertical flanged-wheel type, as much as 80% of the thrust is taken by the front edge under full load. The use of thrust wheels at Norris Dam permitted the earth-fill runway at the tail tower to be built 10 ft narrower than the conventional type would have required. At the head tower (Fig. 1), it would have been dangerous to permit thrust on the front of the runway, first because this was directly on the edge of the bluff and especially because of the trestles at the ends. For these reasons, towers with horizontal thrust wheels at the back of the runways were chosen.

This type of design later proved its effectiveness when a cave-in occurred under the head-tower runway. It was conceivable that at this time one of the cableways might have been damaged considerably had not the horizontal thrust been transmitted to the back of the runway where it was solidly anchored into the hill. The runways were constructed so that all concrete in the main dam, the power house, and the apron could be placed from the cableway. The only place that was inaccessible was the small section of core wall which passed under the tail-tower runway on the left bank. This yardage was small, and special provisions were made for it.

Before the final decision was made other methods of placing concrete were also considered but these were found to be neither as economical nor as practicable for this particular project. The following advantages seemed to justify a cableway installation:

1. Clear access was provided to all parts of the work at all times without the inconvenience of trestles and derricks.
2. A hoisting capacity of 18 tons was available everywhere on the job from each cableway, and a load could be transferred easily from any part of the job to another. Loads up to 50 tons could be handled by using the two cableways and taking special precautions to avoid impact.
3. The penstock sections, power house steel, drum gates, roadway bridge, form panel assemblies, and other miscellaneous equipment could be transferred and set in place without making special derrick set-ups or other special provision.
4. The short span and favorable runway topography brought the cost within economic limits.

Description of Principal Features

General.—Each of the Norris Dam cableways was designed for an 18-ton normal hook load and a 25-ton maximum load under reasonable overload conditions. The main features consisted of a track cable supported at each end on structural steel traveling towers (Fig. 1). The span, center to center of pins, was 1,928 ft 0½ in.; and center to center of back tower legs, 1,925 ft 6 in. The hoisting speed was 300 ft per min; lowering speed, 400 ft per min; carriage traveling speed, 1,200 ft per min; and tower traversing speed, 50 ft per min. Table 1 gives a summary of the principal physical data.

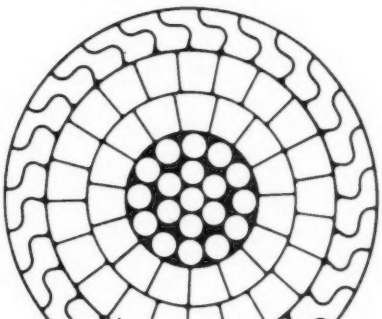


Fig. 2.—Cross Section Through 3-In. Track Cable

Main Track Cable.—The 3-in. diameter locked coil cable (Fig. 2) was socketed at both ends with roller-bearing swivel sockets. At the head-tower end the bearings were seated against a cross-bar which closed the open end of a heavy U-bolt, the closed end of which bore against the main pin in the tower.

TABLE 1.—Physical Data; Norris Dam Cableways

Item No.	(*a*) Towers	Head tower	Tail tower
1	Weight of framing, in tons	126	130
2	Total weight (exclusive of counterweight), in tons	227	180
3	Weight of counterweight, in tons	390	445
4	Vertical height from track-cable connection to track, in feet	75	110
	Distance, Center to Center, of Trucks, in Feet:		
5	Parallel to track	32.05	38.05
6	Perpendicular to track	37.50	55.00
	Counterweight Blocks (Each Block):		
7	Dimensions, in feet	10 × 3 × 2	10 × 3 × 2
8	Average weight, in tons	4.5	4.5
	(*b*) Trucks	Main	Thrust
	Number:		
9	Trucks per tower	4	2
10	Double-flange wheels per truck	8	4
11	Diameter of wheels (double flange), in inches	27	24
12	Weight of rail, in pounds per yard	131	131
13	Distance, center to center of truck wheels, in feet	4.96	3.00
14	Diameter of wheel axles, in inches	7	5.44
15	Weight per truck, in tons	9.1	2.6
	(*c*) Carriage, Fall Blocks, and Fall Rope	Dimensions	
16	Total weight, in tons	11	
	Carriage Sheaves:		
17	Number	12	
18	Tread diameter, in inches	24	
19	Number of hoist-line sheaves	4	
20	Tread diameter of load-line sheaves, in inches	35.01	
	Fall Block:		
21	Number of sheaves	2	
22	Tread diameter, in inches	44	
23	Capacity of hook, in tons	25	
24	Number of parts on fall line	4	

Item No.	(*d*) Head-Tower Machinery	Diameter	Length	Weight
25	Main hoist: Three-drum; all-electric; hoist drums, 49-in. diameter by 44 in.; endless drum, 45-in. diameter by 44 in., with removable spool, 52 in. by 20 in.; nominal capacity, 18 tons	....	....	....
	Weights of Hoist, in Tons:			
26	Endless drum	....	....	12.5
27	Hoist drum	....	....	10
28	Spare hoist drum	....	....	10
29	Stands, etc.	....	....	4.5
30	Bed plate	....	....	5
31	Main hoist motor: 400-hp, 2,200-volt, three-phase, 60-cycle, 600-rpm, wound-rotor, induction motor; total weight (with bed plate), in tons	....	....	5
32	Air compressors: Two compressors per head tower; capacity, 50 cu ft per min; each driven by an 8-hp, 440-volt, three-phase, 60-cycle, motor; total weight (two compressors and receivers), in tons	....	....	2.5
33	Air receivers: Two welded tanks, bell ends; capacity, 24 cu ft; 24-in. diameter by 90 in. long	....	....	....
34	Tower-moving winches: One for each tower; single-drum electric; drum, 21-in. diameter by 26 in., with removable spool 32.5 in. by 12.75 in.; spool driven by 75-hp motor; 440-volt, three-phase, 60-cycle, 900 rpm, wound-rotor induction; weight of winch and motor, in tons	....	....	5
	Miscellaneous Weights, in Tons:			
35	Electrical equipment, head tower	....	....	3
36	Electrical equipment, tail tower	....	....	3
37	Sheaves on top of head tower	....	....	1.5
38	Sheaves on top of tail tower	....	....	1
39	Track cable: locked coil (weight in pounds per foot)*	3	1,900	22.2
40	Endless cable (diameter in inches and length in feet)	1	4,110	....
41	Hoist cable (diameter in inches and length in feet)	⅞	2,500	....
42	Button line (diameter in inches and length in feet)	¾	2,025	....
43	Take-up cable (diameter in inches and length in feet)	1⅜	600	....
	Traversing Cable (Diameters in Inches and Length in Feet):			
44	Tail tower	⅞	2,100	....
45	Head tower	⅞	2,200	....

* 3-in. locked coil; distance between sockets, 1,900 ft.

At the tail-tower end, a similar arrangement was used except that two eye-bolts were used instead of the U-bar. The eye ends of the bolts were connected to the track cable take-up through a pin. The roller-bearing mountings permitted the track cable to be turned so that the wear would be distributed evenly over its entire circumference. When the loaded bucket was being transported the carriage was not run closer to either support than 140 ft, but when the hook was empty the carriage was permitted to approach close to either tower.

The take-up cable for the track cable consisted of a $1\frac{3}{8}$-in. regular lay cable, dead-ended to the top of the tail tower (Figs. 1 and 3). This cable passed

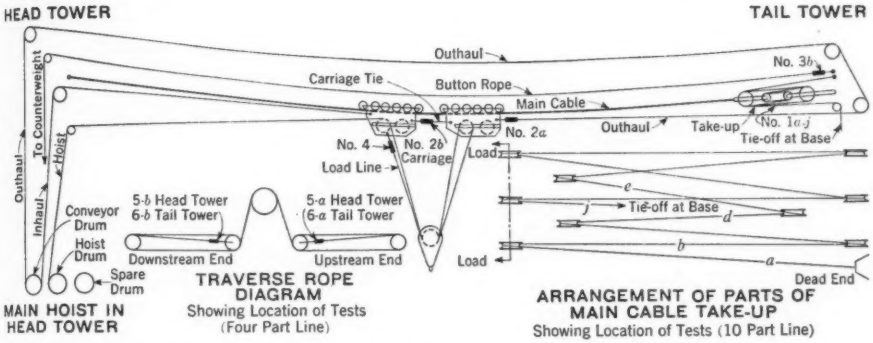


FIG. 3.—ROPE DIAGRAM. NUMBERED AND LETTERED POINTS SHOW LOCATION OF TESTS

through a series of sheaves forming a 10-part line, then back over a sheave in the tail tower and down to the base of the tower where it was made fast. The take-up was fastened to the tower by a main pin and attached to the track cable at the other end, as previously described. The sag of the main track cable, 0.055 of the span, was maintained within 5% by means of the take-up.

Operating Cables.—The endless cable, also called the conveying cable, was of 1-in. diameter plow steel. In the original installation regular lay was used which was later changed to flattened strand due to excessive wear on the conveyer drum and the cable. This cable (Fig. 3) is what its name implies—a continuous line fastened at both ends to the carriage. One part of the cable, called the "outhaul" cable, was attached to the front of the carriage; it passed over two sheaves in the top of the tail tower, then spanned the entire distance to a sheave in the top of the head tower, then down to the conveying drum on the hoist where it made four and a half turns around the drum. Continuing back up to a sheave in the top of the head tower and out to a dead end on the rear of the carriage, this other part is called the "inhaul" cable. Rotations of the conveying drum moved the carriage back and forth on the track cable.

The hoisting cable was of $\frac{7}{8}$-in. diameter improved plow steel. This cable (Fig. 3) was dead-ended on the carriage, passing through the sheaves on the carriage and hook block to form a 4-part line, from which point it was returned over a sheave at the top of the head tower and thence to the hoisting drum. By turning the hoisting drum the load was raised or lowered. The hoisting and endless drums were geared together so that when the carriage moved back

and forth on the track cable the load remained at the same elevation—it was impossible to hoist and convey at the same time.

The button line consisted of a $\frac{3}{4}$-in. cable dead-ended on the tail tower, spanning across to a sheave in the head tower. After passing over this sheave, the cable was counterweighted with a concrete block weighing approximately

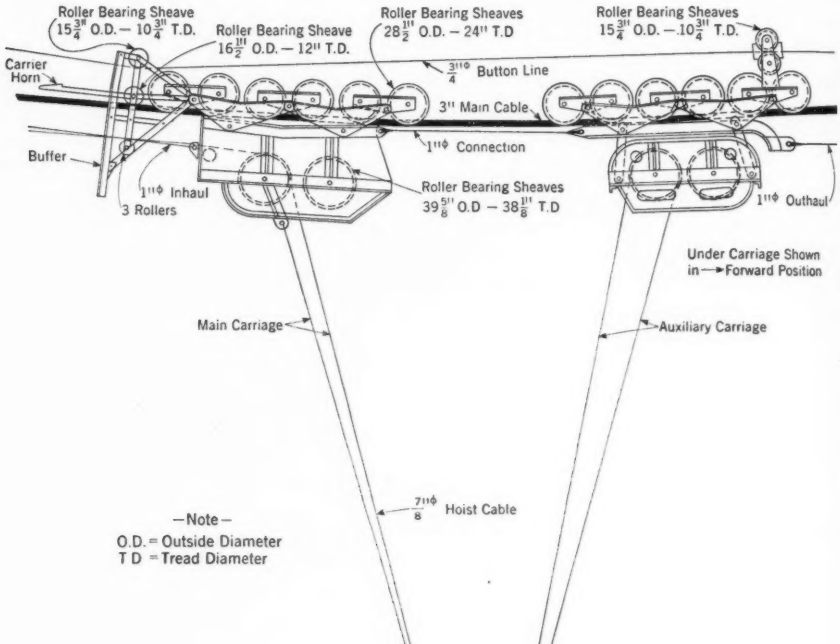


FIG. 4.—CARRIAGE ASSEMBLY

6,000 lb. At regular intervals throughout the span buttons were fastened to this cable. These were so located and were of such a size that each one intercepted a carrier as the carriage passed along the span. These carriers supported the hoist cable and inhaul cable, eliminating deep sags or slack in these lines.

Originally the design of the cableway called for another operating rope for dumping the bucket from the cableway tower. An extra drum in the hoist and extra sheaves in the carriage were provided for this purpose; but this method of

dumping was considered too dangerous for continuous service because it would have been possible to dump a bucket of concrete accidentally in mid-air. Therefore, use of this cable was abandoned, and the extra drum was used instead as a spare hoisting drum. These various ropes adjacent to the towers are shown in Fig. 1 and their relations to the operation of the cableway are indicated diagrammatically in Fig. 3.

Carriage.—The carriage (Fig. 4) was made up of two sections, the main and the auxiliary carriages. Each section was supported on six 24-in. diameter roller-bearing track wheels, equalized to provide as nearly as possible equal loads on the track cable. The carriage supported a hook of 25-ton capacity mounted on a swivel bearing on the fall block which contained two 44-in. diameter sheaves mounted on roller bearings. Each section of the carriage supported a sheave to carry the hoist line. The reason that these sheaves were placed separately in each carriage, instead of side by side in one main carriage, was two-fold: (1) This arrangement spread out the load in such a way that more track wheels could be used, thus reducing the maximum wheel load on the track cable; and (2) it provided a means of preventing the hook and fall block from twisting. Thus the bucket always faced the same way without spinning.

At the rear of the main carriage a carrier horn was provided (Fig. 4) to support the carriers, which in turn supported the operating ropes when the carriage was away from the head tower. The auxiliary carriage was hung from the track wheels on hangers so that the sheaves could be swung backward and forward and locked into position to permit adjustment which would change the position of the resultant load and allow better equalization on the track wheels. Lengthening the 1-in. tie between the carriages, as might be necessary in deep canyons, would require this adjustment.

Towers.—The head tower, shown in Fig. 5, was of heavy structural steel, 75 ft 0 in. high supported at the four corners by spherical bearings mounted on 8-wheel trucks designed to equalize wheel loads (Fig. 6). The horizontal thrust was taken by two 4-wheel trucks placed at the rear corners of the tower shown in the foreground of Fig. 6. These wheels were 24 in. in diameter. The floor framing was designed to carry the machinery loads, comprising the hoist, hoist motor, traversing motor, air compressor, and electrical equipment. The side framing on the lower panel consisted of a steel truss designed to carry the floor loads, the bottom chord extending beyond the back of the tower to form a bracket for supporting the thrust wheels and to provide a platform for the counterweight (Fig. 6).

It will be noted that the entire lower part of this tower was covered with corrugated iron housing. The top supported the main pin holding the track cable, and also the sheaves for the endless, hoist, and button cables. Near the top was a platform for servicing the carriage and the various sheaves. It extended 25 ft each side of the center line and, from its ends, messenger cables, on which 1,000-watt electric lights were strung for job illumination, spanned the valley. The control house, about halfway up the tower, was high enough for the operator to have a clear view of the entire work with the possible exception of part of the loading platform just below the near bank.

The tail tower (Fig. 1) was of similar construction except that the floor loads were much lighter. It was 110 ft 0 in. high. The only machinery in this tower was the traversing motor and winch with its attendant electrical equipment.

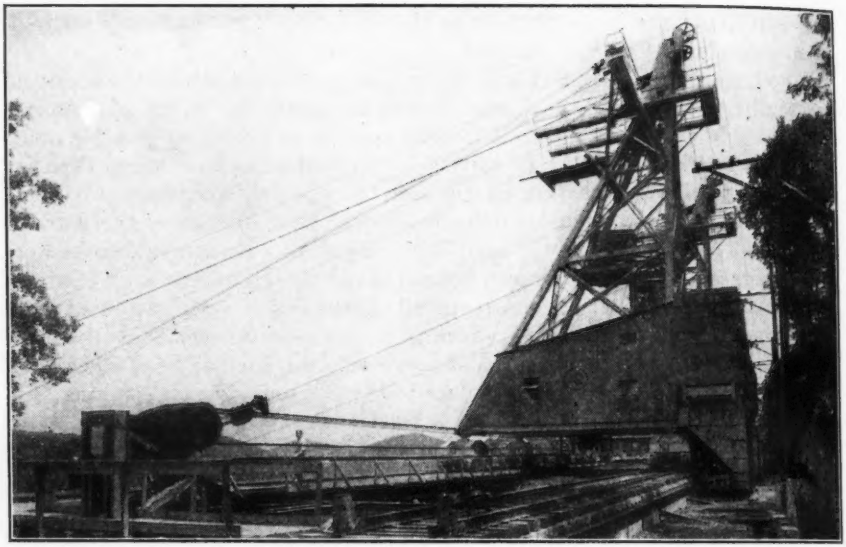


FIG. 5.—GENERAL VIEW OF CABLEWAY HEAD TOWERS, THRUST RAILS, TRUCKS, AND TRAVERSING CABLE

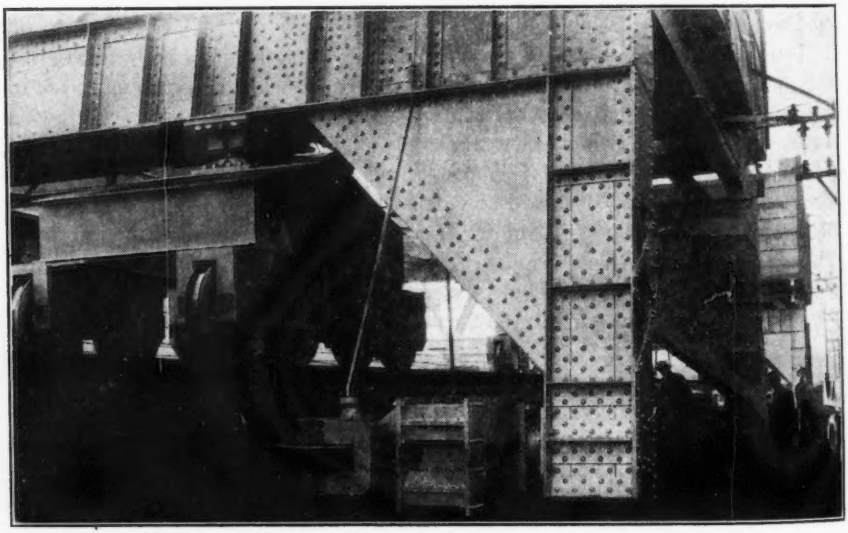


FIG. 6.—DETAILS OF THRUST RAIL AND BALL-AND-SOCKET TRUCK CONNECTION

Counterweights.—Counterweights on these towers, shown in the photographs, rested on two 36-in., 192-lb beams, supported on a bracket made by an extension

of the bottom chord of the truss, located at the rear of the towers. These counterweights differed from some installations because they were made up of individual concrete blocks instead of solid blocks poured in place. It was expected that these towers would be used on future projects, and accordingly an easily removable counterweight was desirable. The head-tower counterweight consisted of 84 blocks with a total weight of 780,000 lb, while that for the tail tower contained 96 blocks weighing 890,000 lb.

Main Hoist.—The main hoist consisted of three drums. It was driven by a 400-hp heavy-duty, wound-rotor motor. The conveying drum lagging was 52 in. in diameter and provided space for several turns of the conveying cable. The hoist drum and the spare drum were each 49 in. in diameter and were grooved for $\frac{7}{8}$-in. cable. The spare drum was permanently connected to the hoisting drum so that the brakes and frictions of both units could be used, thus giving an added safety feature. Heavy-duty service brakes of the band type were provided on each drum for operation in either direction. The brakes were capable of sustaining the load under all conditions of operation, including interruption of power supply. Regenerative braking and provisions for motor plugging served to restrain the lowering of heavy loads. Hydraulic, motor-operated, emergency brakes, which were power released and spring set, were also provided on the hoisting motor for operation in case of power failure or overspeed.

Electrical Equipment.—Both head and tail towers secured their respective power requirements from 2,200-volt, three-phase, 60-cycle trolleys which ran parallel to and at the rear of each runway. Suitable trolley collectors were mounted on a trolley arm as shown in the upper right of Fig. 6.

The incoming line from the trolley arm of the head tower connected through disconnecting switches and terminated on the incoming line panel. This panel consisted essentially of an oil circuit breaker together with outgoing terminals for feeding the primary side of a bank of three 25-kva transformers and main hoist primary panel. Terminals also led from the 440-volt secondary side of the bank and, after connecting through protective devices, fed two air-compressor motors.

The main-hoist primary panel consisted of a magnetically-operated reversing switch with protective devices and outgoing terminals for the main-hoist motor. The maximum running torque of this motor which drove the hoisting and conveying drums was 400% of the 400-hp rating. Each of the three hoist drums was equipped with solenoid valves for operating the brake and clutch. The hoist control was so arranged that under normal operation, with the motor connected electrically to the power lines and the drums clutched to the motor shaft, the speed of the motor would be automatically reduced by regenerative braking if the operator allowed the lowering speed of the load to exceed a certain limit as determined by the setting of the overspeed switch with which the hoist motor was equipped. The secondary panel for the hoist had a number of magnetic contactors for cutting in or cutting out resistance in the rotor circuit. Rectifiers provided direct current for the coils of the solenoid valves and the accelerating relays of hoist and travel control.

The traversing motor was similar to the main hoist but operated at 75-hp, 900 rpm, 440 volts, with a maximum running torque of 400% of the 75-hp rating. It was controlled by the traversing control panel which consisted of a magnetic reversing switch, with protective devices and contactors. Power was supplied from the 75-kva transformer bank. The traversing motor was provided with a thruster brake, and when the motor was de-energized the brake set, thereby bringing the tower to a standstill.

Master controllers were provided for both head and tail towers in the control room of the head tower, thereby giving the operator complete control of both towers. Limit switches regulated travel of the tower in each direction, and prevented skew operation between the head and tail towers beyond a certain limit. The tripping of one of these switches resulted in disconnecting power from the traversing motor and the application of its brake. A separate transformer supplied power for lighting in the head tower.

The electrical equipment in the tail tower was similar to that of the head tower, except that only the incoming line panel and traversing control panel were provided. Either tower could be moved at the will of the operator in the head tower. An overhead control cable connected the head and tail towers.

Operation and Maintenance

Method of Operation.—The system of operation of these cableways is commonly known as "full-view operation," as distinguished from "blind operation." The operator in the control cab (Fig. 1) was able to see all the work and observe the performance of the cableway in handling its load. He also had the further advantage of seeing the tail tower. All operations were by remote control from the cab on the head tower.

Instructions from two signalmen were received by telephone through a loudspeaker in the cab. One signalman was at the loading point to control the loading and dispatching operations and the other at the unloading point. All three men were in two-way communication at all times. Every operation was ordered by a signalman and the operator took matters in hand only in emergencies. The value of the visual control system in preventing serious accidents was demonstrated several times on this job.

Mixed concrete was brought to the cableways in transfer cars, running on a standard-gage track extending the full traverse of the cableways, and thence was dumped into cableway buckets resting on a loading platform which extended the full length of the transfer tracks. This arrangement permitted the buckets to be filled at any point along the cableway traverse. Each car, carrying 6 cu yd, required from 18 to 30 sec to dump, depending upon the slump of the concrete.

For delivering concrete a bucket of special TVA design (Fig. 7) was carried on the hook continuously. It was a bottom-dump type, weighing about 7,000 lb. Its capacity was $7\frac{1}{2}$ cu yd, and normally it carried 6 cu yd of concrete. Particularly noteworthy is the fact that the entire bottom of this bucket drops away for quick emptying.

To cause the filled bucket to swing clear of the transfer tracks the carriage was run a few feet toward the tail tower just before hoisting. The bucket was

then hoisted high enough to clear all obstructions and the carriage was run out to a spot over the point of deposit. There the carriage was brought to a stop so as to eliminate swing of the bucket. After lowering and dumping, the bucket was again raised to clear obstructions and the carriage returned to the head tower. This cycle could be completed to any spot within range of the cableway within 4 min. At close range, it was possible to effect a 2-min. cycle. Operation of the two cableways, 3 transfer trains, and 3 mixers were coordinated by a dispatcher stationed in the mixer plant, making it possible to supply concrete of different mixes at different points at the same time with very little difficulty.

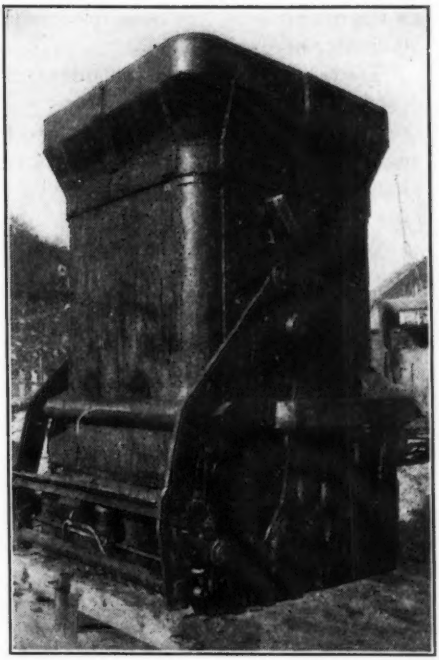


FIG. 7.—AIR-DUMP 6-YD BUCKET DEVELOPED AT NORRIS DAM

During peak production, four crews worked five and one half hours each, although the cableways operated twenty-three hours per day, one hour being taken at noon for inspection and greasing. In addition to the general cableway foreman, the normal operating crew necessary for two cableways was as follows: Two operators each shift, one head-tower oiler each shift, one tail-tower oiler each shift, four signalmen each shift, one foreman 8 hr per day, and one foreman 6 hr per day. The foremen acted as spare operators when necessary.

Most of the operators had had previous cableway experience before their employment at Norris Dam. The signalmen and the oiler in the head tower were riggers who showed special ability at their respective jobs, and the tail-tower oiler was a semi-skilled laborer. Each of the foremen was a former operator.

At first the 6-yd square buckets were not satisfactory because several men were needed to dump them. Air operation of the dump was devised because of the speed and saftey, and this was used successfully throughout the job. Considerable study was made to lessen the "bounce" of the bucket when it was dumped. Finally an inverted V-shaped baffle was installed across the center of the bucket which slowed up the discharge sufficiently to reduce the "bounce" to a minimum.

Cable Maintenance.—The largest recurrent repair item was replacement of the operating cables. Whenever possible, this was undertaken on Sundays, but in many cases changes had to be made during normal working hours. At first these changes took considerable time, but later they could be made in an average

of one hour for button or hoist cable and three hours for endless line. Cables were inspected and greased each day during peak operation but later they were inspected every second day and greased once a week. While this maintenance and replacement could be made with four riggers, often more were used to speed up the operation. These men formed a separate rigger crew and were used for all similar operations on the job.

The track cable was turned one eighth of a revolution about its horizontal axis every six weeks to present a new rolling surface to the carriage wheels. At first the button lines failed because of excessive wear just in front of the buttons, apparently caused by the slack carrier rebounding after striking the button and hitting the cable. This was overcome to a considerable extent by slipping a pipe sleeve about 18 in. long over the cable in front of the button.

When both the cableways had operated a total of 16,947 hr the record of cable replacement was as given in Table 2. After the cableways had been

TABLE 2.—RECORD OF CABLE DURABILITY

Cable	Size and type	Number replaced	Operating hours each	Material cost per operating hour
Button	¾-in. regular lay	10	534.3	$0.591
	¾-in. flattened strand	5	1,586.8	0.286
Endless	1-in. regular lay†	7	1,149.3	0.795
	1-in. flattened strand	4	1,467.6	0.888
Hoisting	⅞-in. regular lay	16	887.8	0.519
	⅞-in. flattened strand	1	806.5	0.851
Traversing (new only)	⅞-in. 6 by 19 regular	6	1,936.0	0.333
Take-up	1⅜ in. 6 by 19 regular	2	5,625.0	0.053
Endless drum wearing plates*	With regular lay	6	567.6	0.577
	With flattened strand‡	2	2,478.6	0.129

* Partial record. † Does not include one cable that broke after 86 hr of service. ‡ Note the increased life of the wearing plates with the use of flattened strand.

operated for a sufficient period, discarded endless and hoist lines were used to replace worn traversing cables.

TABLE 3.—QUANTITIES OF MATERIAL, OTHER THAN CONCRETE, HANDLED BY CABLEWAYS

Item No.	Material	In tons
1	Construction machinery	210
2	Dam closure operation	223
3	Dismantling concrete, No. 3 cofferdam	3,850
4	Drum-gate steel and machinery	680
5	Excavation	22,500
6	Form hoists	11,250
7	Form lumber	3,400
8	Grout cement	4,840
9	Penstock steel	586
10	Pipe, conduit, etc.	500
11	Power house permanent equipment	1,760
12	Power house structural steel	810
13	Reinforcing steel	2,800
14	Riprap, east embankment	8,190
15	Road material and maintenance	875
16	Spillway bridge steel	375
17	Spillway tube gates, castings, and operating equipment	1,087
18	Tile and rock for drains	900
19	Timber cribs for cofferdams	4,375
20	Tractor gates and operating machinery	385
21	Trashrack steel	575

Performance.—Until April, 1936, which was practically at the end of concreting operations, concrete had constituted about 2,000,000 tons or 96.96% of the tonnage moved by cableway. The estimated tonnage of other equipment and materials handled is as given in Table 3.

Material which came in by truck was unloaded at a road which extended to the base of the dam next to the power house on the left bank of the river. For such unloading the cableways traveled downstream from the dam. Since the carpenter shop was located within the range of the cableways, forms built there could be moved directly to the point at which they were to be used.

The maximum placement of concrete in a single month was 92,780 cu yd for both cableways in April, 1935. In this period the average hourly placement was 157.3 cu yd. The maximum 24-hr placement was made on January 7, 1935, when 4,090 cu yd were placed for an average of 170.4 cu yd per hr. The maximum rate in a single shift was made July 10, 1935, when 1,445.8 cu yd were poured between 4 p.m. and 12 p.m. for an average of 180.7 cu yd per hr. Under perfect conditions one cableway placed 162 cu yd per hr, but normally the continuous rate was limited to approximately 120 yd per hr.

Three 3-cu yd concrete mixers, each operating on a 3-min cycle, limited concrete placement to 180 cu yd per hr. It was the practice to stagger the time required for handling other material such as lumber, reinforcing steel, or forms by the cableway throughout a shift, because one cableway could be spared for short periods and return to clear the accumulated concrete from the loading track without delaying the mixing plant. The same condition was encountered in changing from one form to another. One cableway carried more than its share of the concrete until the crew which had changed to another form was ready to begin placing at the new location.

In summary form, Table 4 indicates the quantities of concrete and other materials handled. As noted, delays amounted to 9.3% and 10.9% of the gross operating time for cableways Nos. 1 and 2, respectively. During the period covered by Table 4 each cableway consumed an average of 66.2 kw of electricity per gross operating hour, exclusive of lighting.

Dual Use of Cableways.—Although each cableway operated separately under ordinary conditions, both were used for placing loads in excess of the capacity of one cableway but within the combined capacity of both. This arrangement was used to place the following material:

Material	Weight, in tons
16 penstock sections, 20 ft in diameter by 20 ft long	36–40
2 power house crane girders, 62 ft 3 in. long	33
1 generator rotor	35
8 generator stator frame sections	30
2 turbine water wheels	51
6 spillway bridge girders, 107 ft long, 7 ft 1 in. deep	40

As an example of how heavy loads were handled by two cableways, Fig. 8 shows a 40-ton bridge girder being carried from a trailer at the concrete-loading platform to its final position over the spillway. For this work a 36-in.,

175-lb lifting beam, 29 ft 6½ in. long, weighing 6,800 lb, was used. At each end 6-in. by 1-in. bars were fastened to engage the shackles from the cableway fall blocks. At the center a 4-in. eye-bolt, with a nut swiveling on a roller

TABLE 4.—NORRIS DAM CABLEWAY PERFORMANCE JULY 17, 1934, TO APRIL 1, 1936

Description	Cableway No. 1	Cableway No. 2	Total
(*a*) TIME OPERATED, IN HOURS			
Elapsed	14,496	14,399	28,895
Idle	3,713	4,243	7,956
Gross operating	10,783	10,156	20,939
Delays			
Mechanical	285	374	659
Cable replacement	147	120	267
Waiting	390	438	828
Maintenance	176	175	351
Total	998	1,107	2,105
Net operating	9,785	9,049	18,834*
Placing Concrete			
Dam	6,774	5,451	12,225
Power house	230	387	617
Core wall	229	154	383
Miscellaneous	62	93	155
Total	7,295	6,085	13,380
Total other work	2,490	2,964	5,454
(*b*) QUANTITIES HANDLED			
Concrete, in Cubic Yards			
Dam	555,460	408,011	963,471
Power house	12,485	11,482	23,967
Core wall	8,292	2,845	11,137
Miscellaneous	1,603	4,752	6,355
Total concrete	577,840	427,090	1,004,930
Other work, in tons	35,010	33,272	68,282
Concrete in Cubic Yards per Net Hour			
Dam	82.0	74.9	156.9
Power house	54.3	29.7	84.0
Core wall	36.2	18.5	54.7
Miscellaneous	25.9	51.1	77.0
Average for all concrete	79.2	70.2	149.4
Other work in tons per net hour	14.1	11.2	25.3

* The total net operating hours at the close of the job was 20,918.

bearing, turned in a bolster which was clamped to the beam with four 2-in. rods. In order to strengthen the beam against lateral deflection, four 1-in. rods trussed the top and bottom flanges.

A 30-in. pendulum, with a target, was attached near the center of the beam to assist signalmen in keeping the beam level. One signalman controlled both cableways and it was necessary to plug in his telephone at three different points

during the travel of the load. About four hours were required to transfer the first girder because some time was lost in checking the sag of the main cables and in maneuvering the trailer on the loading platform. The remaining girders required approximately two hours each to place.

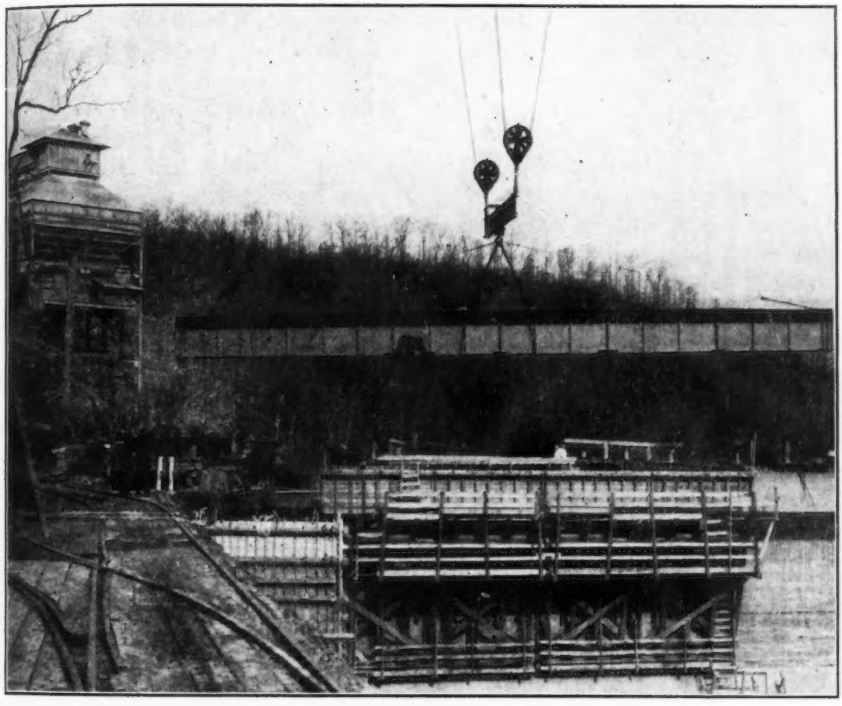
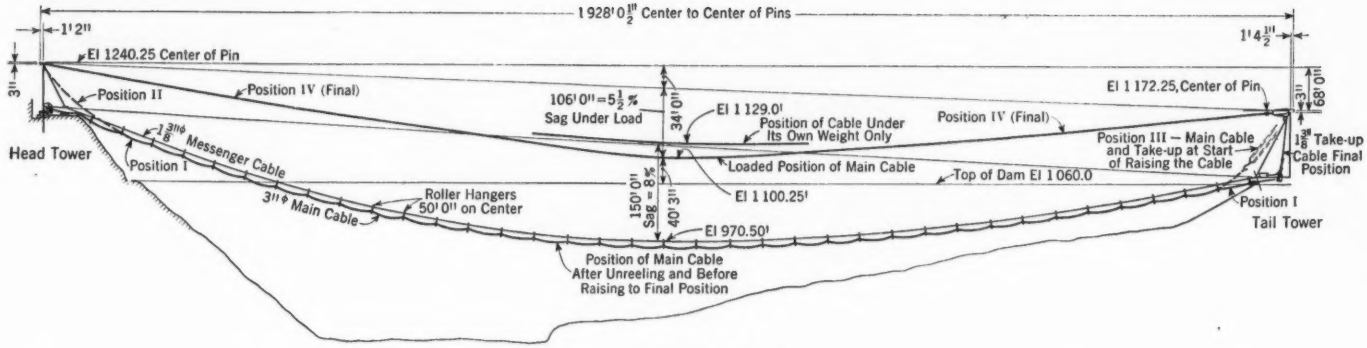


FIG. 8.—TYPE OF HEAVY LOAD HANDLED BY BOTH CABLEWAYS WORKING AS A UNIT. FORTY-TON GIRDER FOR SPILLWAY BRIDGE IS BEING PLACED

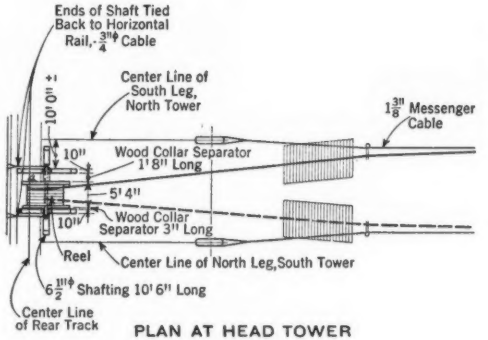
The turbine water wheels constituted the heaviest units handled by the cableways. This load was lifted about 1 ft and carried about 18 ft utilizing the lifting beam just described. The 48-ton wheel together with the 6,800-lb lifting beam made a total load of 51 tons, or 25½ tons per cableway.

TRACK CABLE ERECTION AND DISMANTLING

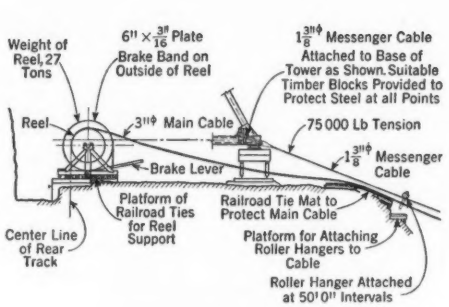
Erection.—The track cable was erected according to the plan and sequences shown in Fig. 9. A 1⅜-in. messenger cable was strung across the valley with its ends fastened to the lower front corner of the towers. The cable reel was set up on a temporary support located between the two head towers. An improvised brake band was placed on two flanges of the cable reel. The cable was first unreeled to bring the socket to the edge of the cliff where a special hanger was attached. This hanger was approximately 6 ft long; it consisted of a single roller on the 1⅜-in. messenger cable with an attachment at the other end for clamping to the cable socket, to prevent the cable from rotating. The track


1 928' 0½" Center to Center of Pins
1'2"
1'4½"
El 1240.25 Center of Pin
3"
Position II
Position IV (Final)
106' 0" = 5½ % Sag Under Load
34' 0"
El 1 129.0'
Position of Cable Under Its Own Weight Only
Position IV (Final)
El 1 172.25, Center of Pin
3"
68' 0"
Head Tower
1⅜" ϕ Messenger Cable
Position I
Loaded Position of Main Cable
Position III — Main Cable and Take-up at Start of Raising the Cable
1⅜" Take-up Cable Final Position
Top of Dam El 1 060.0
3" ϕ Main Cable
Roller Hangers 50' 0" on Center
150' 0" Sag = 8 %
40' 3"
El 1 100.25'
Position I
Tail Tower
El 970.50'
Position of Main Cable After Unreeling and Before Raising to Final Position

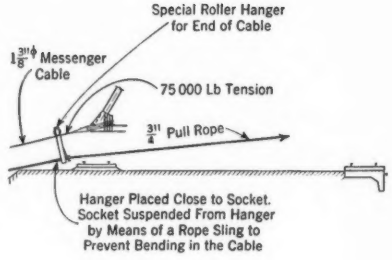
PROFILE OF CABLEWAY


Ends of Shaft Tied Back to Horizontal Rail, ¾" ϕ Cable
10' 0" ±
Center Line of South Leg, North Tower
1⅜" Messenger Cable
10"
Wood Collar Separator 1' 8" Long
5' 4"
10"
Wood Collar Separator 3" Long
Reel
Center Line of North Leg, South Tower
6½" ϕ Shafting 10' 6" Long
Center Line of Rear Track

PLAN AT HEAD TOWER


Weight of Reel, 27 Tons
6" × ¾" Plate Brake Band on Outside of Reel
1⅜" ϕ Messenger Cable Attached to Base of Tower as Shown. Suitable Timber Blocks Provided to Protect Steel at all Points
Reel
3" ϕ Main Cable
75 000 Lb Tension
1⅜" ϕ Messenger Cable
Brake Lever
Platform of Railroad Ties for Reel Support
Railroad Tie Mat to Protect Main Cable
Center Line of Rear Track
Platform for Attaching Roller Hangers to Cable
Roller Hanger Attached at 50' 0" Intervals

SECTION AT HEAD TOWER


Special Roller Hanger for End of Cable
1⅜" ϕ Messenger Cable
75 000 Lb Tension
¾" Pull Rope
Hanger Placed Close to Socket. Socket Suspended From Hanger by Means of a Rope Sling to Prevent Bending in the Cable

SECTION AT TAIL TOWER

FIG. 9.—ERECTION PLAN FOR MAIN CABLE

cable was then pulled out on the messenger cable by a ¾-in. cable spanning the valley. At intervals of 50 ft small single-roller hangers were attached.

When the track cable had been strung completely across the valley (position I, Fig. 9), the socket at the head tower was raised to the top of the head tower and attached to the U-bolt on the main pin (position II). The take-up assembly, which had been previously attached to the main pin of the tail tower, was next attached to the other end of the cable. The take-up was then pulled up (position III), raising the track cable sufficiently to hang by its own supports in order to release the roller hangers. These were taken off by a rigger who went out over the span in a specially devised bosun's chair. Then the cable was raised to its final position (position IV) with the take-up.

Dismantling.—The dismantling of the cableway was somewhat simpler because it was possible to lower the track cable to the roadway across the top of the dam. It was then disconnected from the towers and reeled on a special reel mounted on a trailer, beginning at the tail-tower end. About four hours were required to set up the equipment, and about sixteen hours for reeling. When the cable was reeled the drum contained 5 full layers and 2 turns of the sixth. The weight of the cable, with its sockets, was about 24 tons, making the total weight of the loaded reel 26½ tons. A tractor hoist driven by a 20-hp electric motor and mounted on a 40-ton trailer was used to turn the reel and pull the trailer.

Service Test.—Before the cableways were placed in operation a series of load tests were made. The test load was conveyed, lowered and raised, and the towers were traversed. The brakes were severely tested by allowing the load to run away and be caught by the brakes. Safety devices were checked, and the entire installation carefully inspected.

Design

Specifications.—The original specifications for the Norris Dam cableways were prepared by the Bureau of Reclamation and contained the following paragraphs relative to design:

> The towers shall be of structural steel of liberal design to carry the maximum working tension of the track cables, plus the tension due to the hoisting, dumping, conveyer and carrier ropes, plus a horizontal wind load of 30 lb per sq ft of gross area, plus the dead load of the counterweight and cableway machinery, plus such other loads as the builder's design may impose. The design shall have a factor of safety of not less than 5, plus an allowance for impact stresses produced by ¼ of the lifted load. Members subject to a reversal of stresses shall be designed for the maximum stresses with 25% added for end connections. Except as otherwise specifically stated herein, the design shall be in accordance with the "Specifications for Steel Head Frames and Coal Tipples, Washers and Breakers," as given in the "Structural Steel Handbook," by the late Milo S. Ketchum, M. Am. Soc. C. E., except that the working load stresses shall be in conformance with the current "Standard Specifications for Highway Bridges and

Incidental Structures" of the American Association of State Highway Officials.

The carriages and fall blocks shall be designed for maximum hook loads of 18 tons on either hoisting or dump blocks, with a factor of safety of not less than 5. The carriages shall be made in two sections to permit spreading of fall lines when operating from the maximum hoisting distance. The track rollers shall be not less than 24 in. in diameter with machined grooves having a depth of not less than three quarters of the rope diameter. Each roller shall develop a load of not to exceed 5,000 lb on the track cable. Proper provision shall be made for equalization to avoid excessive loading at any point.

The size of track cables shall be based on a factor of safety of 3, as shall be determined by test, which shall take into consideration the maximum hook load, weight of carriage and fall blocks, ropes, horizontal component due to tension, and any other factors that may produce stress.

Subsequently, an addendum was issued by the TVA in which the following statement was added:

> The weight of the counterweight shall be determined by increasing by 25% the balancing force required under the condition of lifting 25 tons with a 5½% sag on the track cable.

The specifications called for a span of 1,898 ft with a track cable deflection under load of 5½%. Later, the span was lengthened to 1,925.5 ft.

Conditions Used in the Design Check.—The towers were checked for the conditions of loading and stress given in Table 5. The impact is defined as the additional stress produced by the effect of vertical oscillation of the track cable. This is most pronounced when a concrete bucket is dumped or when a bucket which is being raised or lowered is stopped suddenly.

The Lidgerwood Manufacturing Company, American Steel and Wire Company, and the Virginia Bridge Company collaborated in the design of the towers and equipment, and the resulting design was checked by the Construction Plant Division of the TVA. Wind load on exposed areas did not control except in certain bracing members. Unless this stress amounted to more than 25% of the combined dead-load and live-load stresses it was neglected.

For the loads listed in Table 5 the entire horizontal component was considered acting at the base of the tower through the horizontal thrust wheels. The design of the towers presented no special structural problem after the loads were computed. In checking the design the condition with load at midspan gave the maximum stress in the cable and therefore in the towers. Stresses in the track cable and operating cables were also computed with the load at the loading point, and at 140 ft from the tail tower. These two conditions gave maximum stresses in the outhaul and inhaul cables but did not affect the tower design.

Track Cable Stress.—In calculating the stress in the track cable, the formulas adopted were those ordinarily used, as follows:

TABLE 5.—DESIGN FACTORS FOR TOWERS

Loadings	Items	Allowable Stresses
	(*a*) CASE 1	
Dead load	Tower, machinery, counterweight	Tension: 13,000 lb per sq in.
Track cable pull (5½% sag)	Weight of cable, carriage, hoist, blocks and bucket plus 18-ton live load	Compression: $\frac{13{,}000}{1 + \frac{L^2}{13{,}500\, r^2}}$
Operating cable pulls	Conveyer, button, hoist	
Light cable pull	2 cables at 30,000 lb each	Factor of safety = 5
	(*b*) CASE 2	
Dead load	Tower, machinery, counterweight	Tension: 16,000 lb per sq in.
Track cable pull (5½% sag)	Same as Case 1 adding 25% impact to 18-ton live load	Compression: $\frac{16{,}000}{1 + \frac{L^2}{13{,}500\, r^2}}$
Operating cable pulls	Conveyer, button, hoist	
Light cable pull	2 cables at 30,000 lb each	
Wind	30 lb per sq ft on exposed area	Factor of safety = 4
	(*c*) CASE 3 (CABLES OFF)	
Dead load	Tower, machinery, counterweight	Tension: 16,000 lb per sq in. Compression: $\frac{16{,}000}{1 + \frac{L^2}{13{,}500\, r^2}}$ Factor of safety = 4
	(*d*) CASE 4 (EXTREME CONDITION)	
Dead load	Tower, machinery, counterweight	For this condition 10% increase in stress over Case 2 allowed
Track cable pull (5½% sag)	Same as Case 1 with 25-ton live load	
Operating cable pulls	Conveyer, button, hoist	
Light cable pull	2 cables at 30,000 lb each	

NOTES.—

Maximum $\frac{L}{r}$

100 for main compression members
140 for secondary compression members
200 for secondary compression members

Allowable Stresses
Shear on rivets = 10,000 lb per sq in.
Bearing on rivets = 20,000 lb per sq in.
Shear on beam webs = 8,100 lb per sq in.

For load at midspan (see Fig. 10(a)):

$$d = (d_1 + d_2) = \frac{h}{2} + 0.055\, s$$

$$t = \frac{s\,(2\,W + w\,s)}{8\,d - 4\,h}$$

$$y = \frac{w\left(\frac{s}{2}\right)^2}{8\,t}$$

$$\tan \beta_1 = \frac{\frac{h}{4} + \frac{d_2}{2} + 2\,y}{\frac{s}{4}} \qquad \tan \beta_2 = \frac{\frac{d_2}{2} - \frac{h}{4} + 2\,y}{\frac{s}{4}}$$

$$T_1 = \frac{t}{\cos \beta_1} \qquad T_2 = \frac{t}{\cos \beta_2}$$

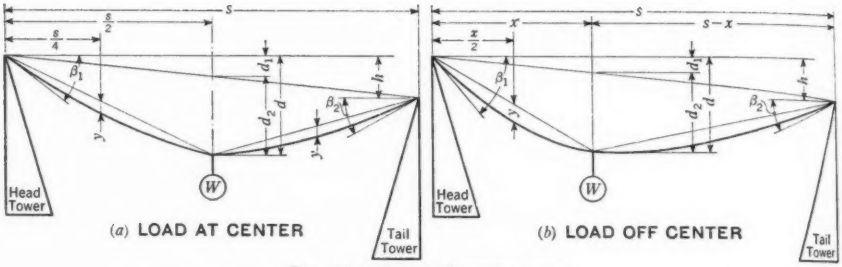


FIG. 10.—DESIGN NOMENCLATURE

For load at points other than midspan (see Fig. 10(b)):

$$d = (d_1 + d_2) = \frac{x\,h}{s} + \frac{x\,(s - x)\,(w\,s + 2\,W)^2}{2\,t\,[w\,s^2 + 4\,W\,\sqrt{x\,(s - x)}]}$$

$$t_1 = \frac{w\,x\,(s - x)}{2\,d_2} + \frac{W\,x\,(s - x)}{s\,d_2}$$

$$y = \frac{w\,x^2}{8\,t_1}$$

$$\tan \beta_1 = \frac{d + 4\,y}{x}$$

$$T_1 = \frac{t_1}{\cos \beta_1}$$

The following nomenclature and data were used, in addition to those shown in Fig. 10:

W = total concentrated load in pounds (58,000 lb including carriage, fall block, cables, etc.);
w = weight of cable in pounds per foot = 22.2 lb;
s = span in feet = 1,925.5 ft;
h = difference in elevation of supports in feet = 68 ft;
d = total deflection at load point from highest support in feet;
x = distance from highest support to the load in feet;
t = horizontal component of cable tension in pounds with load at center;
t_1 = horizontal component of cable tension in pounds with load at a point other than center;
T_1 = cable tension at high tower in pounds;
T_2 = cable tension at low tower in pounds;
y = deflection at center of one-half span or part span in feet;
β_1 = angle cable makes with the horizontal at high support;
β_2 = angle cable makes with the horizontal at low support.

Curves showing the computed maximum stress for a 17-ton load, with and without impact as computed by these formulas, are shown in Fig. 11 (curves *A*, *B*, and *C*). Original computations gave a stress of 362,000 lb with an 18-ton load at midspan without impact. The curves are drawn for a 17-ton load in order to be comparable to the stress tests which were made later under an actual load of 17 tons. A 3-in. locked coil cable which has a tested breaking strength of 1,100,000 lb, giving the proper factor of safety of 3, was used.

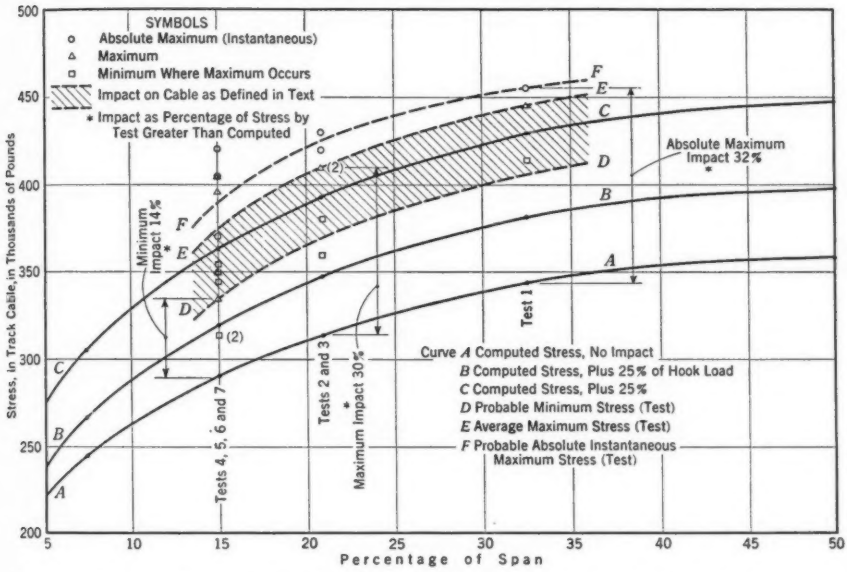

SYMBOLS
Absolute Maximum (Instantaneous)
Maximum
Minimum Where Maximum Occurs
Impact on Cable as Defined in Text
* Impact as Percentage of Stress by Test Greater Than Computed
Stress, in Track Cable, in Thousands of Pounds
Percentage of Span
Absolute Maximum Impact 32%
Minimum Impact 14%
Maximum Impact 30%
Tests 2 and 3
Tests 4, 5, 6 and 7
Test 1
Curve A Computed Stress, No Impact
B Computed Stress, Plus 25% of Hook Load
C Computed Stress, Plus 25%
D Probable Minimum Stress (Test)
E Average Maximum Stress (Test)
F Probable Absolute Instantaneous Maximum Stress (Test)

FIG. 11.—COMPARISON OF COMPUTED AND OBSERVED STRESSES IN MAIN CABLE

Carriage and Operating Cables.—The stresses in the conveyer cable were determined by a graphical analysis of the carriage in order to determine the variation in the wheel loads and also to check the conveyer cable pull. The carriage was assumed to be in a rest position at three points—midspan, loading point, and 140 ft from the tail tower. With the carriage at rest the resultant is perpendicular to the track cable. The result of this computation was a carefully designed carriage which fixed the position of the hoist sheaves in both the auxiliary and main carriages in such positions that the resultants passed as nearly as possible through the centers of the carriages. It is not possible to maintain this condition throughout every position on the cableway span, but it was possible to keep the individual wheel loads within the specified 5,000 lb. A series of computations for the pull of operating cables gave the data shown in Table 6.

Light and Conductor Cables.—For the tension from the light cables and conductor cables it was assumed that there was a $5\frac{1}{2}\%$ sag under their own dead weight. The extent and number of light cables was not known definitely at

first. Due to this uncertainty the total pull of light and conductor cables was assumed at 60,000 lb. A final check reduced this to 32,300 lb.

TABLE 6—MAXIMUM PULL ON TOWERS FROM OPERATING CABLES (OCCURS WHEN LOAD IS IN CENTER)

Cable	LOAD, IN POUNDS		
	At loading point	At midspan	140-ft from tail tower
	(*a*) HEAD TOWER		
Button	6,000	6,000	6,000
Outhaul	13,000	13,000	19,000
Inhaul	14,500	1,200	0
Hoist	9,900	9,900	9,900
Dump*	970	970	970
Total	44,370	31,070	35,870
	(*b*) TAIL TOWER		
Button	6,000	6,000	6,000
Outhaul	{13,000 13,000	{13,000 13,000	{19,000 19,000
Total	32,000	32,000	44,000

* Later omitted although included in computations.

Counterweight.—The counterweight was calculated exactly as detailed in the specifications. It equaled 25% more than a weight sufficient to balance the tower with a 25-ton load at midspan.

Wheel Loads and Reactions.—A summary of the tower reactions and wheel loads is given in Table 7. The maximum wheel load occurs when all the cables are off. This amounts to 77,500 lb. However, this condition never occurred because the full counterweight was not placed on the tower until after the main cable was in place. Actually the maximum wheel load, amounting to about 66,000 lb, probably occurred when the cableway was lifting a 25-ton load. The vertical wheels were crane type, rolled steel, 27 in. in diameter.

Temperature Effect.—The effect of temperature on the stress of the main cable was carefully checked after the cableway was in operation. The sag of the cable was checked for the unloaded cable at various temperatures, and a curve showing sag against temperature was drawn (Fig. 12). A corresponding curve showing the sag of the loaded cable plotted against temperature was calculated, as shown in Fig. 12. The stress in the cable for both loaded and unloaded conditions was then plotted over a range of 100° F. It will be seen that the change in stress due to 100° F was the difference between 340,000 lb and 375,000 lb, which is approximately 10%. With a factor of safety of 3 for the cable and the liberal factor of safety of 5 on the towers, this variation due to temperature could safely be neglected provided the sag of the cable was maintained within the limits of the contract, namely, 0.055 of the span plus or

minus 5%. In summer the cable had to be raised so that the correct sag occurred at about 75° instead of at 33°.

TABLE 7.—Loads and Reactions on Cableway Towers, in Thousands of Pounds, Summarized from Original Computations

Loads and reactions	Loading Conditions				All cables off-dead load of tower and counterweight only
	Dead-weight cable only; no live load	Dead-weight cable and carriage plus 18-ton live load, at center; 5½% sag	Dead-weight cable and carriage plus 18-ton live load plus 25% impact†; 5½% sag	Dead-weight cable and carriage plus 25-ton live load, at center; 5½% sag	
Head Tower:					
Main cable pulls*	163	396	446	460	....
Light and conductor cable pull	....	60	60	60	....
Counterweight	780	780	780	780	780
Tower, trucks, and machinery	453	453	453	453	453
Total Tower Reactions:					
Front track vertical	348	932	1,020	1,058	12
Back track vertical	895	360	280	245	1,203
Horizontal	161	451	500	514	....
Wheel Loads:					
Front vertical	22	58.2	64	66	0.750
Back vertical	56	22.4	17.5	15.3	75
Horizontal	20.1	56.4	62.5	64.3	....
Tail Tower:					
Main cable pull*	162	393	442	456	....
Light and conductor cable pull	....	60	60	60	....
Counterweight	890	890	890	890	890
Tower, trucks, and machinery	361	361	361	361	361
Total Tower Reactions:					
Front track vertical	329	902	1,000	1,018	−5.8
Back track vertical	930	375	282	250	1,237
Horizontal	160	448	496	510	....
Wheel Loads:					
Front vertical	20.6	56.3	62.5	63.6	−0.360
Back vertical	58.2	23.4	17.6	15.6	77.5
Horizontal	20	56	62	63.7	....

* Includes 31,000 lb pull from operating ropes (button, conveyer, hoist, and dump).

† Impact in these computations was taken as 25% of the 18-ton load.

Traversing Force.—The force necessary to traverse the tower was arbitrarily assumed to be 80,000 lb, which was the expected pull exerted on the anchors at either end of the runway. This is a highly indeterminate figure and depends on the smoothness of the track, friction of the wheel bearings, and positions of various loads on the tower. It also varies with the length of time the tower remains standing still, and is usually considerably higher when starting than when actually traversing. Experience on the Madden cableway dictated the choice of 80,000 lb. Test results, as shown subsequently, indicate that this was a fair value.

Design of Runways.—Track for the vertical wheels was designed to take the heaviest wheel loads, and consisted of the following: 131-lb A.R.E.A. Rail and 7 by 9 in. by 8 ft oak ties 18 in. on center. The rear track was supported on a concrete base designed with a vertical cantilever on the rear side. This

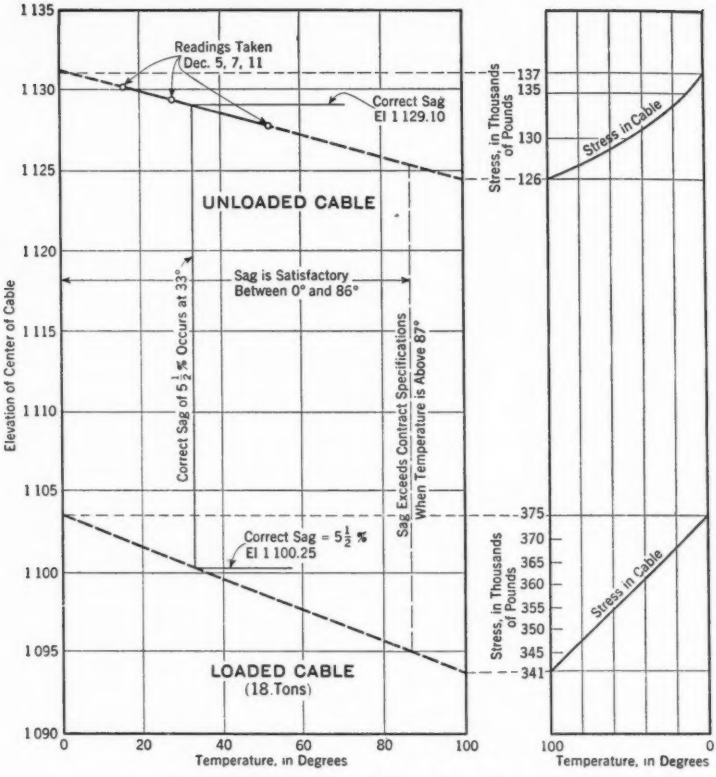

Readings Taken Dec. 5, 7, 11
Correct Sag El 1 129.10
UNLOADED CABLE
Sag is Satisfactory Between 0° and 86°
Correct Sag of 5½% Occurs at 33°
Sag Exceeds Contract Specifications When Temperature is Above 87°
Correct Sag = 5½% El 1 100.25
LOADED CABLE (18 Tons)
Elevation of Center of Cable
Temperature, in Degrees
Stress, in Thousands of Pounds
Stress in Cable

FIG. 12.—TEMPERATURE STRESSES IN MAIN CABLE

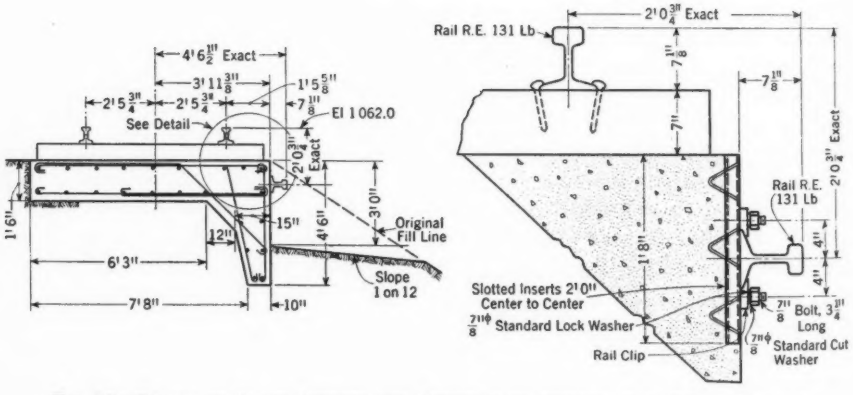


FIG. 13.—DETAIL OF TRACK SUPPORT AND THRUST RAIL SUPPORT, TAIL TOWER RUNWAY

base transmitted the horizontal thrust as well as the vertical load into the earth fill as shown in Fig. 13. The horizontal thrust rail was also a 131-lb rail. It was supported directly against the concrete face, as shown, and was fastened to the concrete by $\frac{7}{8}$-in. bolts and standard rail clips. The bolts were attached to the concrete by slotted inserts which were placed vertically and spaced 2 ft on centers along the face of the concrete base. These inserts provided a means of adjusting the thrust rail vertically to maintain alinement in case the concrete base should settle unevenly. This precaution proved to be justified in the tail-tower runway because in some places the fill settled as much as 6 in. The main track was blocked up with an auxiliary concrete slab and the thrust rail was adjusted accordingly. The front track was laid on stone ballast.

The tail-tower runway was uniform throughout, but the head-tower runway offered some difficulties. The center section was built in a manner similar to the tail tower, except that part of the concrete base for the rear track was placed directly on rock. At each end of the head-tower runway it was necessary to provide a trestle (Fig. 1), consisting of a heavy steel structure designed for three conditions of loading: (1) 18-ton live load plus 25% impact; (2) dead load only; and (3) 25-ton live load, allowing 10% overstress. Horizontal thrust was transmitted to the foundation by footings of L-shaped section, one leg vertical. The anchorage support for the sheaves, which carry the traversing cable for moving the towers, was provided at each end at the center of the runway on the trestle. At the tail tower suitable concrete anchors were provided at each end of the runway to support these sheaves.

Tests

In order to confirm design assumptions, several tests were made and considerable data secured which, in general, check the results given by the design formulas. In particular, they point the way to certain precautions to be taken in the application of these formulas.

Tests with a heavy-duty tension indicator were made to determine actual stresses in the button cable, the outhaul cable, the carriage tie, the hoisting cable, the tower traversing cables, and the 10-part take-up cable, the latter being translated into actual stresses in the main track cable. The general rope diagrams showing the location of the tensometer for various tests are indicated in Fig. 3, and the location of the blocks or sections of the dam to which the cableways were delivering concret ewhen the tests were run is shown in Fig. 14.

The tensometer does not measure stress directly, but, by a diaphragm and pressure gage system, measures the force required to produce a particular deflection in a given length of the stressed cable. The pressure gage readings are reduced to stresses by separate calibration, by loading a specimen of the cable with the tensometer attached in a tension machine and observing simultaneously the various loads applied and the corresponding gage readings. It was found that the precision attained in using the instrument depends largely on the accuracy with which field conditions are reproduced in the laboratory calibration, and tests made at the start of the work indicated that the use of standard cali-

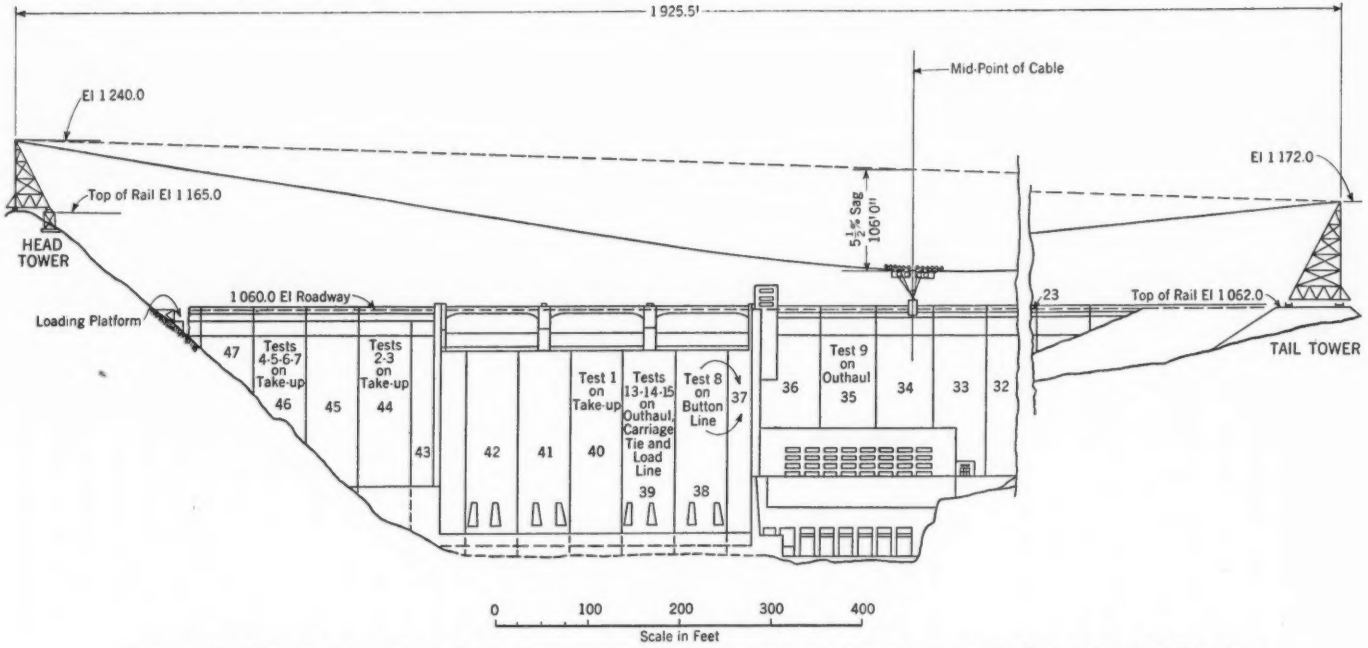


FIG. 14.—DOWNSTREAM ELEVATION OF NORRIS DAM, SHOWING BLOCK NUMBERS AND LOAD POSITIONS FOR ROPE TESTS (SEE FIG. 3)

bration curves would not give the accuracy desired. Accordingly, the field tests were made first, followed by the laboratory calibration.

The tensometer consists essentially of the diaphragm and gage system, a saddle which carries the deflected cable and transfers its load to the diaphragm, and a pair of clamps fixed to the frame of the instrument which bend the stressed cable over the saddle.

Under a rapidly changing load, or when the instrument is not constantly accessible, the clamps cannot be adjusted to maintain a fixed pressure on the cable for all loads. For this reason the clamps were always tightened to the proper degree when the instrument was attached to the field cable and were not altered during the test; a calibration curve was then obtained from a test specimen of the same type of cable with the field conditions duplicated in the testing machine.

The tensometer was equipped with both a visual and a recording gage, the latter carrying circular charts which make one revolution in 30 min. Since the stresses in the cables vary rapidly when the load is in motion, the recorder was used for all tests. The visual gage only was used in the calibration, so corrections were applied to the recorder readings due to difference in elevation of the two gages, difference in the zero readings, and differences inherent in the two gages.

To correct for temperature changes and to detect loss of fluid from the diaphragm and gage system, the instrument was checked at 30-min intervals against a rigid standard bar. The effect of vibration was observed rather closely, and it was concluded that the rapid fluctuation of the record pen was due to variation in the cable stresses rather than to vibration of the measuring equipment. A study of the various sources of errors indicates that the accuracy of the stress measurements under these rather adverse conditions is well within 10%.

The tests were run over complete cycles of operation of the cableway bucket. One cycle consisted of raising the bucket from the loading point, moving it out over the dam, lowering, dumping, raising, moving back to the loading point, and lowering the bucket to its original position. All the cycles were timed independently from the ground and this time record of the cycle was later transposed to the recorder charts. Occasionally a change in reading on the charts located a change in operating cycle closer than these transposed lines of the independent timing.

A complete test at any one point was made up of several runs over several cycles. It was found that the cycle on these runs showed very close uniformity regarding time, there being only a few seconds difference on each run. The time cycle could be averaged within the limits of accuracy, and the stresses in the track cable and hoist cable could be plotted against time. Readings for each position in each cycle were also averaged after correction, and the average cycles were then plotted in diagram form as indicated in Fig. 15. The average minimum, average maximum, and absolute instantaneous maximum stresses were included in plotting the different positions on the cycle.

In Fig. 11 the stresses in the track cable at the points in the cycle where they were maxima, which in each case was the position of lowering the bucket to dump, were plotted and compared with those determined theoretically for this location on the span, using the same formulas and assumptions which formed the basis for the original computations.

Tests on Take-Up Cable.—Perhaps the most consistent and interesting results and undoubtedly the most important were on the take-up tests.

Four tests were run while carrying concrete to block 46, two to block 44, and one to block 40 (Fig. 14). The tensometer was placed in 5 of the 10 take-up parts (Fig. 3) during these tests so that conditions of the tests were sufficiently divergent to establish the consistency of the results. The general cycle of these tests (see Fig. 15) established that, for several seconds after the uphaul begins, the stress in the cable increases very little over that obtained by virtue of the dead loads; and that no impact is apparent. Immediately thereafter came a sudden fluctuation of the recording needle, indicating increase in stress and the application of impact. The explanation of this action seems to lie in the fact that when load is first applied to a cableway, the first few seconds are consumed in readjusting the shape of the main cable to the new sag while the load still retains contact with the ground. Once contact with the ground is removed, impact sets in.

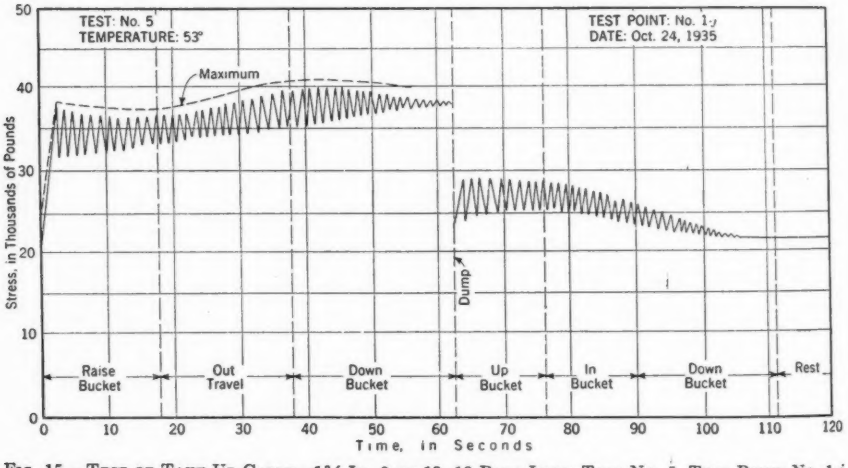


FIG. 15.—TEST OF TAKE-UP CABLE—1⅜ IN. 6 BY 19, 10-PART LINE; TEST NO. 5, TEST POINT NO. 1-*j* (SEE FIGS 3 AND 14)

During the remainder of the uphaul, outhaul, and down travel, the stresses in the main cable increase, but with slightly diminishing impacts. The dumping point in each case is most clearly defined by a sudden stress drop with impact corresponding to a rebound. As might be expected, these stresses decrease during the return travel, and, after the return uphaul, impacts also show decreasing increments until the bucket returns to the loading point. In studying Figs. 15 and 16 it should be recalled that the load is a 6-cu yd bucket weigh-

ing 3½ tons when empty and 17 tons loaded. Since only one cable of a 10-part line was tested, the plotted stresses should be multiplied by 10 to give the stress in the track cable, neglecting friction. Fig. 16 shows a typical chart taken from the tensometer, and the cycles can be traced easily from this chart. No clearly defined variation was apparent in the stresses obtained from the different positions of the gage on different parts of the take-up line. It was found impractical to evaluate any difference in stress in these parts which might lead to a conclusion as to friction values in the sheaves.

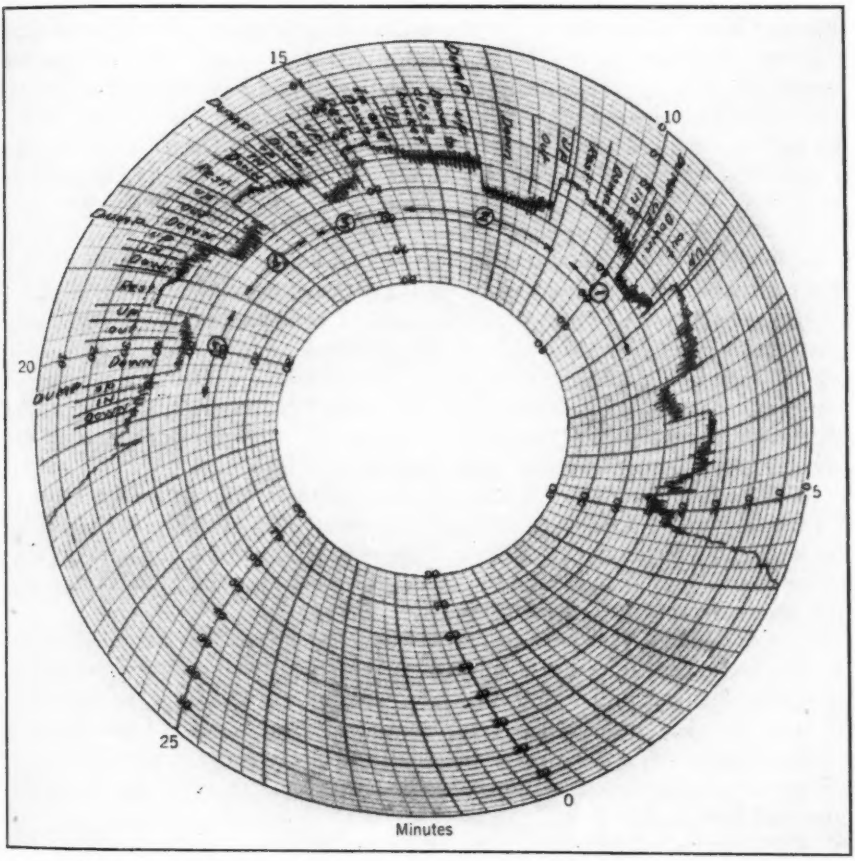


FIG. 16.—TEST OF TAKE-UP CABLE. TYPICAL CHART FROM TENSOMETER; SAME RUN AS IN FIG. 15

In Fig. 11 the theoretical stresses in the main track cable were plotted for any point in the cable with the concentrated load at that point; also the theoretical stresses were plotted under the same conditions with a 25% increase in the load to allow for impact. A similar curve of these same stresses plus 25% of the stress as an impact allowance was plotted. The actual values as determined from the tests were then plotted on this same chart. It is signifi-

cant that for those areas covered by the tests the computed stress plus 25% impact remained within the maximum and minimum actual stress values and closer to the former. The allowance of a 25% increase in the load for impact, as originally computed, is slightly low, and a more accurate figure for impact appears to be 25% of the computed stress.

One interesting disclosure was noted—instantaneous impacts, recorded by the needle jumping up and immediately returning, amounted to about 8% over the more stable maxima.

Other Cable Tests.—At various points indicated in Fig. 3, other readings were taken. Three tests (No. 2*a*) were made on the outhaul cable while pouring concrete in blocks 35 and 39 (Fig. 14). Although these tests were not as consistent as those for the main track cable, they did indicate points of maximum load and impact and prove that this cableway function produced a sizable stress larger than that assumed. Explanation could well be that the sag of the outhaul cable on its clear span from tower to tower was originally estimated at 5½%. As this interfered with the main cable, it was reduced to a point where fluctuations did not cause interference. Consequently, its stress was increased correspondingly. The largest stresses and impacts seem to occur when moving the inertia of the carriage—that is, during carriage travel.

Hoist cable tests (4-part line) in general showed a cycle coinciding closely with those on the main track cable. Stresses in the hoist cable (point No. 4 in Fig. 3) increased rapidly in raising the concrete bucket. Maximum impacts, however, occurred only during carriage travel. An interesting feature was that except for intensities, the same shape of curve for impacts and stresses was obtained for a 3-cu yd load and the usual 6-cu yd load.

Traversing Cable Tests.—Perhaps one of the most difficult design assumptions to confront the designer of a movable heavy-duty cableway is the pull on the anchorages in moving the towers. The tests on the traversing lines brought out some very interesting results. The upstream traversing anchorage and traversing ropes may be seen in Fig. 5.

For designing these anchorages, a pull of 80,000 lb was assumed, based mainly on experience rather than calculations. Under all ordinary conditions this seemed sufficient. The head tower, counterweighted, weighed 1,229,000 lb, and ran on 32 wheels plus 8 thrust wheels. The weight of the counterweighted tail tower was 1,253,000 lb, and it ran on the same number of wheels. No precise distribution of friction in the various wheel sets, nor of the 18-ton live load and 11-ton carriage and fall blocks, was available.

Tests conducted with the cableway loaded indicated 56,000-lb pull on the anchorage required to begin movement of the head tower from rest and 44,000 lb to move the tail tower under similar conditions of loading. With the cableway unloaded, movement of each tower required 44,000-lb pull on the anchorages. It is not likely that any satisfactory explanation for the discrepancy between these two tests exists, because small differences in elevation and alinement in the traversing tracks, a skewed position of the cableway, or the position of the live load may affect this movement. These tests, at points 5 and 6 as indicated in Fig. 3, were conducted during normal operations of the cableway.

Impact stresses of approximately 25% resulted, and the largest variation in stresses occurred when the initial inertia of the tower was overcome to start the tower in motion, after which the stress curve flattened out.

Another test was conducted on the tail tower after it had stood idle for two hours, and with the cableway empty. A maximum pull of 70,000 lb was required to start movement of the tower from rest under this condition. Unfortunately no test was made on the head tower for a similar period of rest. In any case, it appears that the original assumption of 80,000 lb was adequate and justified.

A summary of the theoretical and actual pulls on the head and tail towers from the various operating ropes is given in Table 8.

TABLE 8.—THEORETICAL AND ACTUAL STRESSES IN OPERATING CABLES (IN POUNDS); MAXIMUM PULL AT THE TOWERS

Cables	Load at Loading Point		Load at Midspan		Load 140 Ft from Tail Tower	
	Computed	Observed	Computed	Observed	Computed	Observed
			(*a*) Head Tower			
Button	6,000	7,000	6,000	9,000	6,000	....
Outhaul	13,000	17,000	13,000	27,000	19,000	36,000
Inhaul	14,500	....	1,200	....	....	....
Hoist	9,900	12,000	9,900	16,000	9,900	....
Dump	970	....	970	....	970	....
Total	44,370	....	31,070	....	35,870	....
			(*b*) Tail Tower			
Button	6,000	7,000	6,000	9,000	6,000	....
Outhaul	13,000	17,000	13,000	27,000	19,000	36,000
	13,000	....	13,000	....	19,000	....
Total	32,000	....	32,000	....	44,000	....

Costs

Combining the various expenditures for installing the two units of these cableways, the following costs were determined:

Preliminary work, clearing	$ 2,961.96
Constructing runways	34,993.40
Steel trestle	19,440.26
Thrust anchors	16,584.67
Tracks	13,193.27
Towers, first cost erected	100,318.00
Counterweights and miscellaneous tower expense	14,687.85
Hoists, track cable, carriage, operating cables, electrical equipment and controls, traversing equipment erected	187,613.75
Dismantling	22,591.12
Sub-total	$412,384.28

Salvage value (credit) from transfer of one cableway to another project and the sale of part of one cableway	96,795.79
Net cost	$315,588.49

These data do not include the cost of reconstruction required by the cave-in of the runway foundation in May, 1935, which amounted to $63,723.25.

Likewise unit operating costs on an hourly basis were computed, based on a total of 19,342.5 net operating hours for the two units, between July 17, 1934, and April 1, 1936. This number of net hours is more than in Table 4 because 508.5 delay hours are included. Subdividing into the component parts gave the following:

Operation		
Labor	$5.463	
Expense—electricity, plant, supplies, etc.	1.024	
Total operation, per hour		$ 6.487
Maintenance		
Labor		
Field	$2.508	
Shop	0.274	
Total maintenance labor	2.782	
Expense—shops, machines, etc.	0.658	
Material (original cables not included)	$2.324	
Credit—salvaged cable	0.331	
Net total—materials	$1.993	
Total maintenance, per hour		5.433
Total operating cost, per hour (depreciation not included)		$11.920

This total cost was for the period extending beyond the completion of the dam. The actual cost at the close of the job was $11.44 per hr, which does not include the cost of repairing a cave-in under the head-tower runway.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FUNCTIONAL DESIGN OF FLOOD CONTROL RESERVOIRS

Discussion

BY EDWARD SOUCEK, JUN. AM. SOC. C. E.

EDWARD SOUCEK,[8] JUN. AM. SOC. C. E. (by letter).[8a]—A very convenient method for analysis of the action of flood control reservoirs with fixed outlets has been given by the authors. The selection of an equivalent uniform flood to represent a given non-rectangular inflow hydrograph may prove troublesome, despite the assurance of the authors that this operation is not difficult. A 40-hr uniform flood of equal total inflow is proved to be "equivalent" to the triangular flood shown in Fig. 1 for a particular outflow rate but no basis for the pre-selection of this duration is given. It seems that errors in estimates of the duration of the rectangular flood considerably exceeding those of 5% which are used in Table 1 are easily possible.

It is true that the paper deals primarily with the use of the rectangular flood—not its determination—so if it were not for the example given, this question would not be raised. The writer believes that the latitude which is possible in the establishment of the equivalent uniform flood is reduced if its total inflow is made equal to that of the hydrograph which is to be replaced only up to the time of maximum outflow. The inflow rate after this instant may remain constant indefinitely or may drop immediately to zero without effect upon either the maximum outflow rate or the maximum storage volume. It appears that the exclusion of an extraneous part of the inflow hydrograph makes the selection of a rectangular flood easier.

Considering the triangular hydrograph shown in Fig. 1, if a maximum outflow rate of 5,160 acre-ft per hr is contemplated, the maximum storage and outflow rate occur 32.8 hr after the beginning of the flood. The total inflow up to this time is 253,000 acre-ft. The remaining 40,000 acre-ft of inflow and the corresponding 15.2 hr of time are neglected. By plotting a mass curve of the given flood, the writer found that the least deviation of accumulated

NOTE.—This paper by C. J. Posey, Jun. Am. Soc. C. E., and Fu-Te I, Esq., was published in October, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[8] Asst. Hydr. Engr., The Panama Canal, Balboa Heights, C. Z.

[8a] Received by the Secretary November 20, 1939.

inflows between a rectangular hydrograph and the given inflow hydrograph resulted when the duration of the uniform flood was 31 hr and the intensity was 8,160 acre-ft per hr. By the application of Table 2, the maximum storage for this rectangular hydrograph was found to be slightly more than 116,000 acre-ft, a close check on the step-method solution. This elementary procedure may merely prove the contention of the authors that the selection of the equivalent uniform flood is easy but is intended to substantiate a preference regarding the total inflow of the uniform flood.

In Table 2, the small effect on the detention ratio of considerable variation in the depth-capacity exponent is very significant. Since the point of zero depth is fixed by the elevation of the outlets, a monomial exponential function cannot be fitted to the data as accurately as would be the case if the point of zero depth could be adjusted at will. This restriction is apparently unimportant except for high outflow ratios.

For orifice-type outlets, the depth-discharge exponent may be expected to remain sensibly constant at the value of 0.5. For weir-type outlets, however, the use of 1.5 for the depth-discharge exponent is somewhat in error, owing to the tendency for the constant B in Equation (5) to increase with the head. When only instantaneous discharges are being considered, the use of a constant exponent and a varying coefficient is justified by the resulting convenience of calculation. When a monomial exponential function with both the coefficient and exponent held constant is necessary, it might be better to modify the exponent. Assuming what appeared to be a reasonable variation of the coefficient B with the head, the writer plotted logarithmic rating curves for ogee spillways designed for 5-ft, 10-ft, and 20-ft heads and found that the exponent 1.6 represented the depth-discharge relation better than the exponent 1.5. The effect is smaller than might be anticipated from the variation in B which was used—a linear variation from 3.1 at zero head to 4.0 at the design head. As in the case of the depth-capacity exponent, the difference seems unimportant for low outflow ratios.

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Founded November 5, 1852

DISCUSSIONS

WIND FORCES ON A TALL BUILDING

Discussion

BY J. CHARLES RATHBUN, M. AM. SOC. C. E.

J. CHARLES RATHBUN,[51] M. AM. SOC. C. E. (by letter).[51a]—This paper has been fortunate in having so many able discussers. As many of them have been interested actively in this or a similar problem they have been able to suggest a number of related phases that should be given study. The most outstanding one is that of building vibration.

Strength and stiffness are two entirely different characteristics of a structure, as emphasized by Mr. Coyle and Professor Morris. This paper has been primarily concerned with the lateral loads on a building although it has taken into consideration both of these other questions to a considerable extent. To study the wind effects properly one must consider not only the physical characteristics of the building but also the effect of vibration on the tenants. It was considered unwise to expand the scope of this investigation into this latter field, although the value of it was recognized. Mr. Coyle has collected considerable data on the subject of vibration in various buildings and has given it careful study.

The writer agrees with Professor Morris that extreme refinement of mathematical calculations is unwarranted but that some theory of stress distribution for lateral stability is necessary in designing buildings. A continued study of the problem should reveal better theories that will enable the engineer to anticipate the action of his building under all conditions of loading, without increasing the labor of analysis unreasonably. The writer wishes to emphasize that he did not suggest that the fact—that the architectural clothing adds to the strength of the building—should be taken into consideration in its design. Both conclusions (8) and (9) demonstrate this point; and Professor Morris emphasizes it. The third sentence of Mr. Skinner's discussion is not at all in accord with the writer's opinion; nor does it seem to follow from the findings.

NOTE.—The paper by J. Charles Rathbun, M. Am. Soc. C. E., was published in September, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1938, by Messrs. David C. Coyle, and Clyde T. Morris; January, 1939, by Messrs. Robins Fleming, F. P. Shearwood, Lydik S. Jacobsen, Francis L. Castleman, Jr., J. B. Wilbur, R. D. Spellman, David A. Molitor, Walter J. Gray, and K. L. DeBlois; March, 1939, by Messrs. Albert Smith, and Victor R. Bergman; May, 1939, by Otto Gottschalk, Esq.; and June, 1939, by Colin Skinner, Assoc. M. Am. Soc. C. E.

[51] Associate Prof., Civ. Eng., Coll. of the City of New York, New York, N. Y.

[51a] Received by the Secretary November 1, 1939.

In fact, his account of the gradual transfer of load from the masonry to the steel leads to the opposite conclusion.

Mr. Fleming's discussion is valuable particularly as he gives numerous references on this question and related problems. He quotes the report of the British Research Committee[52]: "In a large class of steel-framed buildings * * * the steel work is thus relieved of the responsibility of resisting wind shear." This idea should be studied carefully and the character of the building noted. For the tall steel building—the skyscraper—the writer feels that the steel should carry the responsibility of the wind load and lateral stability of the structure, even if the load is partly taken by the masonry.

It is true, as Mr. Shearwood suggests, that the data obtained from the Empire State Building show that the problem is far too complicated to be formulated by simple uniform horizontal loads and methods of analysis. In design the engineer endeavors "to distribute, exactly, indeterminate stresses induced by indeterminate forces." The writer hopes that nothing in his paper will lead to the idea that this is not good engineering practice. Of course, formulas must be followed with judgment and experience, not just as a guide. Engineers with sound judgment and broad experience may be able to design safely but most of them should not be encouraged to dispense with accepted methods which in the past have produced safe structures. If the experience and study of the specialists have decided that stresses obtained by conventional methods should not be exceeded is it wise for the Profession to violate these suggestions? In cases where they are sure the previous study does not apply, or where it has been improved upon, a special investigation should be made and the result applied in a conservative manner.

The writer believes (see fifth paragraph in Part II, under "Introduction") that it is probably wise to ignore the strength of the masonry until, as Mr. Shearwood suggests, further investigation of this problem is made.

Mr. Jacobsen has stated the fact (as has the writer preceding and following Equation (1)) that T and W are ratios of time and weight between the building and the model. The facilities for observing the period of vibration by means of the collimator are available for future use. It is planned to make checks from time to time on the building to note if a gradual change takes place in its period of vibration.

It is true that Equation (20) can be written without derivation as it is quite simple. The previous equations were given so that one might note the approximations involved and the effect of these approximations on the validity of this equation. The effect of the walls was omitted because the scope of the study was a comparison between the frame of the building and the model. The results of this study were compared with the actual building and the difference was attributed to the stiffness due to the walls.

The columns of the Empire State Building rest on solid rock whereas the columns of the model are anchored to steel bars. Therefore, both are quite rigid and have little effect in destroying the similitude between model and building frame.

[52] Third and Final Report, British Steel Structures Research Committee, Dept. of Scientific and Industrial Research, 1936.

The writer agrees with Professor Castleman that more data of this nature should be gathered. There are serious difficulties, however. It is seldom that the owner of a building is willing to allow an intensive study of his structure to be made and published. Furthermore, the gathering of these data, their interpretation, the equipment necessary, and its installation, are very expensive.

The writer feels, definitely, that the data obtained from wind tunnels by a skilled observer and under laboratory conditions are most valuable. The data are obtained under controlled conditions and so can be understood much better than those obtained from observations on a building. They are two entirely different sets of data but the reason for the differences between them is worthy of study. The local conditions that tend to break up the wind currents is only one factor, although an important one. The work of Messrs. Dryden and Hill is most valuable and the data therein should be compared with those obtained on the building itself.

The law of variation of pressure with distances from the ground has not been formulated as suggested by Professor Castleman. As can be seen from the data of this paper it is quite erratic. The observations given by Mr. Skinner in his Conclusion (9) are only one set of many and should have no more weight than the other items of Table 2. They certainly do not warrant the authority that is due a "conclusion."

The only objection the writer sees to adding the term suggested by Professor Castleman to Equation (18) is that it increases the length of the equation. When it was omitted it was thought the numerical value would be very small.

The fact that the temperature effect on the building is negligible should not lead one to generalize. The construction of the building, as well as the efficiency of the insulation around the frame, will have much to do with this effect. It was examined into, and recorded, largely to show that it was not the cause of the plumb-bob's failure to return to a fixed center.

Professor Wilbur discusses the deflection index and its importance. The data taken from the Empire State Building should be of considerable value in determining this index.

Mr. Spellman suggests that "in some instances a substantial saving could be attained safely by omitting wind-bracing." This is done in some cities where the ordinance allows one to design buildings less than a certain height without allowing for wind loads. To extend this practice to the top part of tall buildings is an idea of questionable value and one not encouraged by the data of this paper. If there are substantial wind loads the writer feels that they should be provided for in the steel.

It was particularly gratifying to note that the late Mr. Molitor[53] accepted the percentage added to the rigidity by the masonry as obtained by the writer. This value may vary between wide limits even in the same building while the materials are becoming adjusted to each other. He, like the writer, and several of the contributors to this paper seemed to approve the present method of stress analysis and assumed lateral loads. To date the results have proved

[53] David A. Molitor, M. Am. Soc. C. E., died on September 8, 1939.

satisfactory. The tests on the Empire State Building seem to indicate that 20 lb per sq ft is not too great and since there have been few if any failures due to wind it is not too low; and so, until better methods of loading, analysis, and design are discovered, the present method should not be considered outmoded.

Mr. Gray's observation of the fog phenomenon gives some idea of the action of the wind currents about the building.

Mr. DeBlois has expressed an interesting idea relative to the damping effect shown in the model data; and the curves viewed from this standpoint show this effect clearly. The writer was unable to devise any way of measuring this same phenomenon on the building and so could not use it in his paper.

Mr. Smith's plots of wind forces against their direction are very enlightening and, to some extent, show the general direction of the wind in New York, and also the effect of the neighboring buildings. He also calls attention to irregularities in the manometer and extensometer readings. These were noted before submitting the manuscript but it was decided that their omission would tend to lead to a false sense of accuracy.

The extensometer rods were close to the columns and were covered with the same insulating material. Thus the elongation due to temperature will be the same for both rod and column and the reading will not be affected. It has been shown by the test on building deflection that the temperature effect is very slight.

Mr. Bergman emphasizes the importance of rigidity in a building and that the designer is working between limits. As the writer has stated, this is a problem that could be studied to advantage by the Profession.

Mr. Gottschalk's suggestion that there should be more simple methods of obtaining the information about masonry stiffening than by models is one that it is hoped will be acted on and one which will lead to studies on buildings other than the Empire State. The difficulties already outlined by the writer make it probable that not many such studies will be made.

Mr. Skinner is to be commended that his interest in the problem induced him to make further observations after field work was considered finished and some of the instruments had been removed from the building. In many cases, however, his discussion gives an entirely erroneous impression due to an overemphasis on data that are considered by the writer to be of but little moment. This is particularly true of the wind currents in the fire tower and the creaking of the building during a high wind. The actual data on which Mr. Skinner bases his findings were available to the writer when the paper was written.

Mr. Skinner's comments relative to air currents should not lead one to discredit the work of the plumb-bob, and the findings shown in Fig. 12. When the writer first installed this bob it was found necessary to damp it, of course. After this all recorded readings were made with great care and accuracy. Sources of error such as temperature, wind currents, etc., were studied and taken into consideration. At the writer's suggestion, one of the observers made a study of the accuracy of his readings and found that they had a probable error of less than one-eighth inch. This is more accurate than the collimator readings. The eight charts similar to Fig. 26 were a part of a study made

during the early part of the data-gathering period. The idea was abandoned because it was shown that the plasticity of the building is considerable and therefore the center of rest is of no importance.

The creaking mentioned by Mr. Skinner is a very usual phenomenon with structures when new and it would have been surprising if this building had proved an exception. In bridges it is not noticeable because of other noises when the load is being applied; in new occupied buildings this is also true to a limited extent but in new ships and vacant buildings it is noted often. That an observer who is watching for data should note it, is as it should be; that he should pick up the period of vibration from the sound is most interesting and Mr. Skinner is to be congratulated on using this information to advantage. That the noises should be classed as groans is incorrect and misleading. As he stated, when the building reached structural adjustment the sounds abated, just as one would anticipate.

The writer has investigated the question of the partitions that are "seriously cracked" and finds that no reports of any serious, dangerous, or even prominent cracks in partitions or floors have been made to those responsible for the building. As the building is composed of many materials, some weak in tension (concrete and plaster), it would be surprising if some cracks did not appear during the first winter. However, these are very minor and the writer did not consider them of any structural significance except to show adjustment of stress. The partitions are not a part of the working structure and there are no cracks in the floors due to wind.

To be complete, Mr. Skinner's attempt to correlate the observations of the four New York stations should indicate the spread of the data as well as their averages. There are so many factors that enter into this problem that a simple average does not give the true picture; nor can it be applied to one storm, or indicate the maximum velocities at the several stations and their ratios to each other.

The writer agrees with Mr. Skinner and the several other discussers that the work of Messrs. Dryden and Hill is of great value and should be studied by all those interested in the problem. Much of Mr. Skinner's discussion is mainly a repetition of the data, conclusions, and ideas expressed in the paper and with these, of course, the writer agrees. He has tried to give full credit for all of the ideas that were given by those associated with him in the work. However those employed in gathering data were too numerous to mention by name.

Mr. Skinner closes with nine "conclusions." Conclusion (1) is covered by the writer's Conclusion (8). His last sentence depends upon the meaning of "great refinement." In Conclusion (2), the figures given should have been substantiated by data. If they are only a guess the writer questions the wisdom of having given them. His first sentence is a contribution to the subject. Conclusion (3) is a repetition of the writer's Conclusion (2). Since the plumb-bob and Fig. 12 were the sources of the necessary data, and Mr. Skinner questions the accuracy of these, one wonders how he arrived at this conclusion. The writer questions the accuracy of Conclusion (4) but considers

it of little importance. The data from which Conclusion (5) was drawn is probably Table 5. As the vibration and deflection are not simply related phenomena the writer questions the wisdom of generalizing from the data. As the data are available the reader can decide this matter. As for Conclusion (6), a normal amount of creaking and grating can be heard by those who are listening for it but this is not a structural defect. The idea of the masonry slipping on the steel frame is interesting but not probable unless data not published bear it out. The squeaking of joints in metallic partitions has been reported. The building, definitely, does not groan during a high wind. Conclusions (7) and (8) were fully covered by the writer's paper. The information in Conclusion (9) was based on one of the many observations, selections from which were taken to form Tables 1 and 2. The findings scarcely warrant a conclusion, however, as it was only one set of readings.

Mr. Skinner's discussion is full of "food for thought." His observations on the gradual increase of the sensitivity of the steel are of great interest. It is unfortunate that quantitative data were not secured. In many cases his overstatement of the data has led to an incorrect impression but it is probably for the best that a different point of view has been advanced for the reader to consider. The writer has tried not to draw conclusions from general averages of very erratic data but where Mr. Skinner has done so the reader can judge each case on its relative merits since he has all of the pertinent data.

In closing the writer wishes to thank all those who have contributed, either by supplying funds and physical facilities, collecting data, making suggestions, or discussing the paper. It is hoped that future studies will enable the designing engineer to work with greater exactness in predicting the stresses due to wind on his structure because of the ideas and data given in this paper and its discussions.

Corrections for *Transactions:* On page 1358, line 6 following Fig. 14, change "8.38" to "8.28"; and, to the "Acknowledgments" add: "Alfred F. Samuel, Jr., Assoc. M. Am. Soc. C. E., installed the collimator target and made many of the readings."

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DISCUSSIONS

SPECIFICATION AND DESIGN OF STEEL GUSSET-PLATES

Discussion

BY T. H. RUST, ASSOC. M. AM. SOC. C. E.

T. H. RUST,[31] Assoc. M. Am. Soc. C. E. (by letter).[31a]—The discussions on this paper have further integrated the subject. Professor Sherlock and Dean Grinter note the difficulty of the mathematical approach to the accurate stress analysis in steel gusset-plates, and Professor Sherlock states the meager achievements in accurate stress determination of gusset-plates.

The difficulty arises when an attempt is made to state, mathematically, the principle of least energy for a plate with finite boundaries. The forces concerned are those parallel to the plate, applied at points within the plate rather than those applied perpendicular to it. The stress will flow from one force to another along many paths. The amount of stress along a particular path will be a function of its length and the rigidity of the material. Stored energy is a product of force by displacement, or a summation of such products; and the stress in the plate will distribute itself along the various paths in such a manner as to make the total energy in the plate a minimum. A prismatic plate with two opposite forces applied uniformly over its ends will be uniformly stressed similar to Fig. 6(*a*). However, if the forces are applied at points in the ends of the plate, and if the plate is other than narrow in an engineering sense, another component of stress will be introduced from the principle of least energy as shown by Fig. 6(*c*). If there is little bending stress, greater stress will lie in the path directly between the two forces than in any other longer path through the plate. It is this variation of path in stress transfer in gusset-plates from one attached member to another which makes the accurate determination of stress so difficult.

Dean Grinter's calculations of stress in the bakelite models reveal similar difficulties. In Models B_4 and B_5 the computed maximum stress at the edges

NOTE.—This paper by T. H. Rust, Assoc. M. Am. Soc. C. E., was published in November, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1939, by Messrs. R. H. Sherlock, and L. E. Grinter; April, 1939, by Russell C. Brinker, Jun. Am. Soc. C. E.; and June, 1939, by C. W. Wixom, Assoc. M. Am. Soc. C. E.

[31] Structural Engr., Chicago, Ill.

[31a] Received by the Secretary October 23, 1939.

is less than that given by the formula for direct and bending stress. This is because the path in the plate between the ends of the legs is the most direct one and hence takes more least energy stress than the longer paths at the edges. The wider plates of the *D* models show this phenomenon to an even greater degree, the stress at the edges being almost zero. It is well to note that the behavior of stress in bakelite models simulates that of steel in a qualitative, rather than in a quantitative, manner. The fringes fall off more rapidly in the bakelite than they would in a steel plate because the modulus of elasticity of bakelite is only about 550 000 lb per sq in. in comparison with 29 000 000 lb per sq in. for steel. As an extreme example of the localization of stress due to the elasticity of the material, a pin thrust through a thin sheet of rubber could be moved back and forth laterally in the plane of the sheet, producing only local distortions, and points at some distance would suffer little or no distortion. Steel models would not have this quantitative handicap, since all three types of stress (*a*), (*b*), and (*c*), shown in Fig. 6, would appear in their true values if the proper scale relations were maintained between model and prototype. After they are in the plate, the $\frac{P}{A}$-values and the $\frac{M\,c}{I}$-values are independent of the modulus of elasticity of the plate, provided the material satisfies Hooke's law in the region considered. It is only the "minimizing stress" (Fig. 6(*c*)) that is thus affected. Photo-elastic methods could not be used with steel models; but perhaps other methods could be employed such as covering the plates with a thin layer of material which would show changes after the steel plates were subjected to stress.

In the absence of well validated methods for obtaining quantitative information, the qualitative knowledge is a definite help to the careful designer.

Although the mathematical methods are intricate and not far advanced for the study of stress in plates of finite boundaries, with forces applied at points within the boundaries and parallel to the planes of these plates, still progress might be made by methods analogous to those developed by the German physicist, Walter Ritz.[32] He made some noteworthy solutions in thin plates of finite boundaries, with forces applied perpendicular to the planes of the plates, by using variational calculus, polynomials, Green's theorem, and Fourier series. The continual refinement of results by Fourier series parallels the designer's successive approximations in his calculations and designs.

Professor Brinker states that search of the technical literature will disclose few if any instances of failures caused by faulty gusset-plate design. It is indeed unfortunate that more structural failures are not described in the literature, since the primary cause of failure is not always disclosed by a casual inspection afterward. The close study that accompanies the preparation of an engineering analysis and report might very well have placed the blame on these points of discontinuity. Engineers agree quite generally that failures are in the joints and details rather than in the bodies of the members.

[32] "Deformation einer am Rande eingespannten elastischen Platte unter dem Einflusz eines gegebenes Normaldruckes," von Walter Ritz, 1878–1909, *Journal für die reine und angewante Mathematik*, Band 135, 1909, s. 6–57.

Professor Brinker also notes that the gusset-plate should not have an angle of less than 30° with the connecting member. Because of the considerable research that has been completed in riveted joints, gusset-plates, and thin plates,[12, 22] the writer did not include this empirical specification in the paper or in the thesis[16] of which the paper is a part, because of the expectation that it would soon be superseded.

Such matters as thickness of gusset, and reinforcement of gusset to prevent buckling, which Professor Brinker raises should be the result of the specifications and requirements of a particular structure.

Equation (5) has been given in more generalized form, including examples of its application by the late J. A. L. Waddell, Hon. M. Am. Soc. C. E.[15] This formula is approximate and gives only the order of magnitude of plate stresses rather than accurate values.

Mr. Wixom's statement in the first paragraph of his discussion quite agrees with the statement of the writer (see "Recommendations for the Specifications and Design of Steel Gusset-Plates: Stresses"), "If there is no reversal, stressing the rivets beyond the elastic limit is of little consequence." It is this favorable quality in the plastic region of steel that has permitted the specification of gusset-plates to lag in progress behind the specification of members. However, when reversed stresses occur and each reversal goes beyond the slip capacity of the rivets, the connection is very likely to deteriorate.

It seems that it would be desirable in almost every instance to splice all members outside of the gusset connection and thereby avoid superimposing the discontinuity of the splice upon that of the gusset.

The writer agrees with Mr. Wixom that it would be folly to specify a gusset of approximately equal axes at the expense of long rivet connections or unriveted areas of plate, and the writer regrets that such an interpretation has been placed by Mr. Wixom upon the ideal. The ideal gusset-plate is similar to the ideal engineering structure; it can almost never be realized. However, the writer has found, after the examination of many gussets, with the experience and the elementary theory at his disposal before this research was undertaken, that gussets of approximately equal principal axes with forces intersecting near the center practically never showed high stresses due to bending and direct stress. The desirability of avoiding long and narrow gusset-plates has been reported by Clifford E. Paine,[33] M. Am. Soc. C. E.

Mr. Wixom's objection to lug angles is based upon three considerations: (1) Short lug angles induce moment in the transfer of stress; (2) the added rivet connection causes distortion with resultant stress in the lug angle before it can transfer any stress from the member into the plate; and (3) the added material in the lug angle must be distorted with resultant stress before it can transfer

[12] "Tests of Joints in Wide Plates," *Bulletin No. 239*, Univ. of Illinois, Urbana, Ill.

[22] *Proceedings*, Am. Soc. C. E., May, 1939, p. 805.

[16] This paper is a part of a thesis submitted by the writer for a D. Sc. degree at the Univ. of Michigan. A complete copy of the thesis has been filed for reference in Engineering Societies Library, 29 West 39th Street, New York, N. Y.

[15] "Bridge Engineering," by J. A. L. Waddell, 1925, pp. 519–529.

[33] "Designing Bridge Towers 700 Ft. High," by Clifford E. Paine, *Engineering News-Record*, October 8, 1936, p. 499.

stress into the plate. The first consideration can be minimized by long lug angles. The second, which is undoubtedly the most important factor, can be reduced by welding the lug angles to the members. Welded connections are much stiffer than riveted ones[34]; so the less capacity for transmission of stress due to the added riveted connection can be largely counteracted by welding the lug angle to the connecting member. The entire connection from the member and auxiliaries to the gusset-plate, itself, should be either all riveted or all welded. The third factor can be minimized by making the lug angles as narrow as practicable and of comparatively thick material. This factor is not as important as the first and second factors. For these reasons it does not seem justifiable to condemn all use of lug angles.

In answer to Mr. Wixom's preference for heavy trusses, it seems that best results can be obtained by not attempting to exceed, or even equal, the maximum sizes of rivets, heavy plates, and sections in use at this time. At least, this should apply to important gusset-plates and their connections. Many specifications limit the thickness of plates, eye-bars, and other sections for steel structures, and C. F. Goodrich,[35] M. Am. Soc. C. E., reports that "the limit has about been reached in human strength to handle heavier, high-pressure, rivet guns."

In reply to Mr. Wixom's criticism that the models did not simulate closely enough the complex gusset-plates in actual trusses, it may be said that progress in any laboratory work results from simple beginnings. Also the development of a proper testing technique was of major importance in this research. It is true that the stresses are principally direct in most of the models, and in others the bending results entirely from the eccentricity of the direct load, whereas, in actual trusses the presence of diagonals and verticals renders the conditions more complex. Nevertheless, it is believed that the lessons from these simple models have supplied information for the estimation of stress in more complicated joints; and it is hoped that more involved joints will be investigated in the laboratory.

[34] "The Relative Rigidity of Welded and Riveted Connections," by C. R. Young, M. Am. Soc. C. E., and K. B. Jackson, *Canadian Journal of Research*, 11, 1934, p. 131.

[35] "Tension Tests of Large Riveted Joints," discussion by C. F. Goodrich, *Proceedings*, Am. Soc. C. E., September, 1939, p. 1290.

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DISCUSSIONS

THE YELLOW RIVER PROBLEM

Discussion

BY O. J. TODD, M. AM. SOC. C. E., AND S. ELIASSEN, ASSOC. M. AM. SOC. C. E.

O. J. TODD,[32] M. AM. SOC. C. E., AND S. ELIASSEN,[33] ASSOC. M. AM. SOC. C. E. (by letter).[33a]—Since this paper was written, in 1938, there has been a marked change in the lower course of the Yellow River which is no longer flowing in the diked channel north of the Shantung Mountains, but across the plains to the southeastward from a point nearly 40 miles west of Kaifeng. It has joined the Huai River and, together with that river, empties into Hungtze Lake east of the Tientsin-Pukow Railway in Kiangsu Province and from there, via several channels, into the Yellow Sea. Some of the flow follows the course of the Grand Canal to the Yangtzekiang 700 miles south of the 1937 outlet of the Yellow River on the Gulf of Chihli.

This latest great change of course reminds observers of the consequences of a "back to Nature" program. If human beings are going to live and carry on their daily activities in this fertile region, the Yellow River must be held to a channel that is as stable as possible. The inhabitants must have a sense of security if they are to live in peace and build permanent homes. Railroads, motor roads, and canals must all be made secure against serious flood attacks. A section of the Lunghai Railway has already been buried in silt since the dike break of June, 1938. It will not be long until sections of the Tientsin-Pukow Railway in Kiangsu will be similarly threatened. A river which, annually, can fill an area of 300 sq miles with silt to a depth of 5 ft must not be permitted to go uncontrolled across a fertile plain where the average population density is greater than 600 per sq mile.

NOTE.—This paper by O. J. Todd, M. Am. Soc. C. E., and S. Eliassen, Assoc. M. Am. Soc. C. E., was published in December, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1939, by Messrs. J. W. Beardsley, and Elliott J. Dent; April, 1939, by Messrs. Herbert Chatley, and H. van der Veen; June, 1939, by Messrs. C. S. Jarvis, and E. W. Lane; and September, 1939, by George Higgins, M. Am. Soc. C. E.

[32] Cons. Engr., Palo Alto, Calif.; formerly, Cons. Engr. to the Yellow River Comm. and Shantung Govt., Peiping, China.

[33] Oslo, Norway; formerly, Engr. in Chg., Survey Dept., The Yellow River Comm., Kaifeng, Honan, China.

[33a] Received by the Secretary November 13, 1939.

The question may be raised as to which of the two general routes should be chosen as the regulated course of the river—the one north of the Shantung Mountains to the Gulf of Chihli, or the one south of these mountains to the Yellow Sea. Both from technical and social-economic considerations the northern course is decidedly preferable. The distance to the sea is shorter and will remain shorter for the next two centuries even if nothing is done by way of soil-erosion control of the inland areas. Also, the northern route has no complications with other river systems such as would be found along any southern route that might be selected. The northern route is flanked on the south for a distance of 90 miles by the foothills of the Shantung Mountains, making one dike unnecessary in that region; and, since a dike system exists which can be utilized, in part, in a permanent plan of control, it will obviously be less costly to regulate the northern course (of 1851 to 1938) than any southern course, including that being followed since the summer of 1938. The latter would require a new diking system with costly regulating works to control the floods of the rivers draining the southern slopes of the Shantung Mountains, as well as the floods of the Huai River. Since these rivers have serious flood problems of their own, flood control would be complicated by routing the Yellow River that way. The Huai River is notoriously difficult to handle in time of high flood.

The fact that the Yellow River has been so altered in its course since the preparation of this paper does not mean that the problem of control is essentially different from that outlined by the writers. The present (1939) course of the river is likely a temporary one, although it might be maintained to advantage while the former (1851 to 1938) course is being regulated as suggested by the writers. Much of this work could be done "in the dry," at a great saving in costs, and the river then diverted back into the regulated channel from the new (since 1938) course.

Mr. Beardsley asks if it will be practical to fill, with silt, a number of shallow lakes and marshes along the old and former diked courses, and thus convert these areas into good, arable land. This is quite feasible and desirable. The areas are small, however, and confined to narrow strips of land just outside the dikes. During the decade 1929 to 1939, a number of siphons were constructed for the purpose of drawing silty water from the Yellow River in order to irrigate, fertilize, and reclaim just such areas. Experience has shown that these siphons need careful watching in order to avoid drawing in fine sand which may suddenly appear at the surface of the river, due either to small freshets or to the throwing into suspension of bed silt which contains coarser material. Lands have been ruined by careless operation of such siphons. The best success thus far has been obtained along the lower course in Shantung. Several siphons installed in 1936 near Kaifeng had not been in operation long enough to warrant any remarks as to their merits when the war broke out, preventing further experiments.

However, it is the larger aspect of broad-flooding wide areas during floods in order to control the silt that is of major interest. It involves the moving of a population estimated at 200 000 from an area of 400 sq miles, every 8 or 10 yr, into other areas prepared for them, and then fitting the abandoned area

for silt deposition and drainage of the desilted water. The feasibility of such a plan depends very much on the ability of the Government to relocate and raise many towns and villages, giving each suitable protection. Since this process would be continuous and accompanied by constant difficulties with the population, the writers have hesitated to urge the adoption of such methods.

Colonel Dent's analysis of the silt inflow, deposition, and outflow for the Shanchow-Chinchang reach is interesting. However, if there is an excess of silt leaving a reach during a certain period, it is not certain that this excess is due chiefly to tributary inflow. A considerable portion may have been eroded from the river-bed.

The writers agree that for the reach from Shanchow to Mengtsin there will not be much difference in the silt inflow and outflow and that whatever difference there is will be due to tributary inflow; but for the reach from Mengtsin to Chinchang the case is different. Here both deposition and erosion are going on at a rapid rate, especially during the flood season. The net result is a slow but steady rise of the bed and foreshores. Referring to Tables 7 and 8, it will be seen that for the two years 1934 and 1935 the following volumes entered and left at Shanchow and Chinchang (in cubic feet of silt):

Entering at Shanchow	96 174 900 000
Leaving at Chinchang	98 378 890 000
Excess leaving at Chinchang	2 203 990 000

—or, 82 000 000 cu yd. From silt records of the Lo Ho and Chin Ho for 1934 and 1935 (incomplete during the low-water season), it has been computed that these two rivers carried approximately the following silt volumes (in cubic feet):

Year	Lo Ho	Chin Ho
1934	325 000 000	280 000 000
1935	1 750 000 000	770 000 000
Total	2 075 000 000	1 050 000 000

The total of the two is 3 125 000 000 cu ft, or about 115 000 000 cu yd. Taking into account other small streams which brought silt into the reach, one may say that as much as 150 000 000 cu yd of silt entered the Shanchow-Chinchang reach from tributaries during the 2-yr period, 1934–1935. Hence, only 68 000 000 cu yd of silt seem to have been deposited between Shanchow and Mengtsin due to the steep slope. This differs considerably from the value of 500 000 000 cu yd computed by Colonel Dent. Unfortunately, he did not have the Lo Ho and Chin Ho data to guide his judgment.

Colonel Dent finds it difficult to reconcile the 52 000 000 000 cu ft of silt entering the diked section at Chinchang in 1934 (and a similar amount in 1935) with the estimated 22 000 000 000 cu ft passing Litsin at the head of the delta under a regulated condition. Both 1934 and 1935 must be regarded as more than average flow and silt years, however. The yearly average silt volume passing Chinchang has been estimated by the Yellow River Commission at 35 000 000 000 cu ft. Furthermore, the average value of 22 000 000 000 cu ft

passing Litsin was based on the Shantung settling basins being in operation; but it is quite possible that the silt volume reaching the delta every year may be greater than 22 000 000 000 cu ft. If this proves to be the case, the lower course will likely deteriorate more rapidly than estimated. The aim of the regulation must be to attempt as nearly as possible to bring a silt quantity to sea which will bear such a relation to the silt deposition up stream that neither the one nor the other deposition will be out of step and thus unduly accelerate the superelevation of the river's course along that reach.

With regard to the former rise of the river-bed across the plain, the writers have mentioned 3 ft per century as an average value, having checked a similar figure by Mr. Freeman referred to by Colonel Dent. However, this value has been based on the plain outside the dikes as reference; but, obviously, since the plain also has had silt deposited on it, the actual rise, with reference to a fixed plane, must be greater. If, as has been stated, the average yearly silt inflow at Chinchang has been 35 000 000 000 cu ft and the outflow into the delta 15 000 000 000 cu ft, 20 000 000 000 cu ft per yr have been left behind along the diked channel, and on the plain, as a result of dike breaks. The entire river channel has thus risen with the plain, but also slowly above the rising plain.

Assuming that 70% of the silt that has been left behind has been deposited in the up-stream 200 miles of the river's lower course (in the channel and on the plain), there will have been deposited in 100 yr about 1 400 000 000 000 cu ft of silt in this locality. From personal investigation the writers know that practically all of the silt that flows out on to this plain through dike breaks becomes deposited within 30 miles of the dike lines. A section through the deposition normal to the river's course will have approximately the form of a trapezoid, the top being the distance between dikes and taken as 5 miles and the base as 65 miles. Deducting 85 000 000 000 cu ft for the 3-ft rise of the channel over this 200-mile stretch leaves 1 315 000 000 000 cu ft of silt deposited within this section whose height is computed at approximately 6.7 ft. Adding to this the former estimate of 3 ft gives a total rise in a century of nearly 10 ft with reference to a fixed plane. As the deposition is not regularly distributed along the river's course, there will be some variation from this average depth.

It is generally during super-floods, such as in 1933, that vast quantities of silt become deposited on the plain. During that flood alone from 60 000 000 000 to 80 000 000 000 cu ft of silt must have found its way to the plain. Following the break near Tungchuang in 1935, the silt volume deposited on the plain was estimated at 15 000 000 000 cu ft. Added to another large quantity that escaped through breaks at Kuantai the same year, probably as much as 20 000 000 000 cu ft of silt was deposited on the plain outside the dikes in 1935. In 1934, through the Kuantai breach, another 15 000 000 000 cu ft of silt reached the plain. During these three years alone, from 90 000 000 000 to 100 000 000 000 cu ft of silt must have been added to the deposits on the plain. None of this went farther than 30 miles from the dikes.

If the calculation for the rise of the plain and the river channel is further examined, it will be found that only 20% of the volume left behind appears to

be deposited between the dikes, and the remaining 80% is deposited on the plain outside. In view of the foregoing estimates of silt volumes which are known to have flowed out on to the plain, this is not a surprising deduction. It is an approximate calculation, but the result cannot be materially altered even if the dike distance and the deposition limit away from the dike lines are changed 25% either way. It means that approximately 3 000 000 000 cu ft of silt are deposited along the river channel between the dikes every year, and this quantity is a measure for the efficiency of the river channel, as it has existed in the past, to transport silt.

Colonel Dent asks what will happen if the dike distance is narrowed and the dikes strengthened so as to make breaks improbable. Until river-laboratory research work has been conducted to clarify the river's silt action for wide and narrow dike distances, and for different widths of channel under various conditions of discharge, the writers are not prepared to make any definite statement of opinion. It does not seem reasonable, however, to expect a deterioration in the river's capacity to transport silt if the channel is regulated and the dike distance is adjusted so as to give the greatest transporting efficiency for silt. With regard to the 80% silt quantity (which, under existing conditions, is deposited on the plain outside the dikes), the writers have recommended settling basins in Western Shantung, and they have also strongly advocated soil-erosion control measures in the up-stream loess areas in order to reduce, gradually, the silt masses in the river's flow. They realize, however, as suggested by Colonel Dent and Professor Lane, that it may be necessary to have another silt-depositing basin along the up-stream reaches of the river's diked course to be used until the up-stream soil-erosion control program becomes effective. They do not believe, however, that it would be wise, at the outset, to advocate a system of controlled silt-depositing reservoirs on the plain, as this would be likely to stir up strong opposition that would be very difficult to overcome. It could only be done in case the soil-erosion control should prove an absolute failure, and after the delta has advanced so far as to threaten to make the entire lower course untenable. There are social aspects to the problem that add to its complexity.

Colonel Dent wishes a silt-depositing reservoir west of Tungchuang mainly for the purpose of getting rid of the coarser silt particles so that the river channel down stream, where the slope is flatter, will be able to convey the silt. Theoretically, of course, this is correct. The writers deal with this point in their comments on the discussion by Professor Lane and merely wish to state here that west of Tungchuang the natural conditions for silt deposition are not so favorable as east of that place. The silt-settling basins proposed by the writers are so located as to require little construction work to create them and to bring the clarified flow back to the river. A deposition basin, however, is possible immediately east of the Peiping-Hankow Railway, on the north side of the river. The land is low and unproductive at this point, and the population not dense; but costly construction works are required. So very important is the problem of taking care of excess silt that further studies, both in the field and the laboratory, seem necessary for a satisfactory solution.

With regard to the danger of the channel being plugged by silt after a freshet, the writers' opinion is that there may be a danger of plugging during a freshet if the flow is exceptionally silt laden. The high elevation of the old channel prior to 1851, east of Hsüchow, where Mr. Chatley has had occasion to observe it, is probably not due to any plugging after the occurrence of the 1851 breach. Most likely it is due to the same prolonged silt action that has been building up the channels in which the river has been flowing. When a breach occurs, the abandoned channel quickly tends to become choked at that point, and it is only the nearest 15 to 20 miles that are seriously silted up by the rapidly diminishing flow. This was especially noticeable after the Tungchuang breach in 1935. Three months after the break occurred at this point there was almost no flow passing down the abandoned river channel. Cross-sections showed that 15 miles below the break the bed of the channel had retained practically its former elevation and shape.

Mr. Chatley questions the value of 2 500 000 cu ft per sec as a probable maximum for the Yangtzekiang at Ichang, 400 miles up stream from Hankow. One of the writers has had an opportunity to investigate the probable maximum discharge of the Yangtzekiang just below Ichang and has found that it is possible for the aforementioned discharge to occur. The reason the lower stretches of that river do not show a higher discharge is the existence of the two enormous lakes, Tungting and Poyang, which act as regulators for the Yangtzekiang. Usually, when the flood is heavy from the up-stream regions, there are also dike breaks which further aid in regulating the flow. Another moderating factor is found in the fact that the tributaries to these lakes are in flood late in June, whereas, the most serious floods from the main upper river come in late July or August.

In regard to the Yellow River delta surveys, both the British Admiralty and the German land surveys have astronomical positions close to the coast in that region. These surveys have been checked by the more precise surveys made by the Yellow River Commission. Positions of the more important villages have been compared and found to agree reasonably well, especially in latitude. The writers believe, therefore, that the estimate they have made of delta advance is fairly reliable.

Mr. van der Veen complains that the writers have not given due credit to certain previous writers on the Yellow River, notably his countryman, Mr. Fijnje van Salverda, who issued a memorandum in 1887. Lack of space made it impossible for them to devote any considerable portion of the paper to a discussion of former writings, however interesting this might have been. They had so much new material to discuss that they referred the interested readers to Mr. Freeman's paper on "Flood Problems in China"[3] where references to, and discussions of, works on the Yellow River by former writers could be found. The writers wrote entirely from their own knowledge and experience with the river itself and only referred to such works as, to them, seemed of sufficient importance to be of distinct value in adding to the hydraulic knowledge

[3] "Flood Problems in China," by the late John R. Freeman, *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1405.

of the river. They regard the hydraulic laboratory experiments which were made in Germany for the Chinese Government (by a collaboration of Chinese and German engineers under the leadership of Prof. H. Engels), as a distinct contribution toward improving the knowledge of the Yellow River in spite of the fact that they were made without simulating the conditions of the Yellow River too well. When the results of these experiments are compared with those which eminent engineers such as Freeman and Franzius expected from their narrowing in of the dike distance and river channel, then it must be agreed that the experimental results gave "some startling indications."

Mr. van der Veen is correct in urging better dikes and better dike protection for the immediate future, pending a complete solution of the Yellow River problem. This is one of the practical aspects of the situation wherein all western engineers conversant with it agree. In fact, the inadequacy of the dike system, its low factor of safety in many places, and its haphazard location has brought adverse criticism from nearly every investigator who has had the opportunity of making a personal inspection.

The flood problem of the Yellow River does not end merely with strengthening the present dikes, however. To declare that it is merely a diking job is to close one's eyes to the real issue. Mr. van der Veen feels that the writers have explored too many angles of the problem and mentions "seeing a few particular trees but not the forest of which they form a part." He would limit the problem of Yellow River control mostly to a program of strengthening the dikes and banks where the current impinges on them. The writers believe that the problem must be attacked from several directions and that an adequate dike system is only a part of the remedy needed. The Yellow River problem has its roots in the most stupendous and rapid geological transformation process which is taking place on this planet—a process which has been created, for the most part, by human ignorance and carelessness. Soil has been stripped of the cover of vegetation that held it in place, and food-producing plants with less soil binding properties have been put on all these highlands that required careful treatment. The cultivation of these crops induced erosion which has continued without abatement for centuries. To repair the enormous erosion damage that has already been done is impossible; but it may be possible to retard it appreciably, if not entirely to arrest it. This will be the work of decades; and, in the meantime, ways and means must be found to control the floods and the silt masses that make the otherwise attractive and fertile Great Plain of North China such an insecure place in which to dwell. Most likely the Yellow River will always have a silt problem which will be reduced gradually in importance with the development of erosion control. The laws of silt transportation, as they apply to this river, will eventually be determined in suitable field laboratories and will give the river engineer this added power not possessed by the mere dike builder.

The writers have not recommended that dredges be introduced without mature consideration. When the channel has been regulated, the use of dredges may mean much toward a prolongation of its life. How far out into the delta it will be practical to use dredges only experience can prove. This equipment

will have to be of a different type than the antiquated bucket dredge, which, due to bad seamanship, ran on a silt bank, capsized, and was buried in silt at the mouth of the river in the 1890's. That feat should not be cited as a warning to keep dredges from the Yellow River. Unlike the Mississippi, the Yellow River scours on rising stages. If dredged cuts are kept open through crossings immediately before the flood season begins, it may result in the river scouring a channel deeper than otherwise. It is likely that the low-water season's bed-load will be induced to progress down stream more efficiently by dredging cuts through bars which act as obstacles, tending to silt up the channel as a whole. A judicious use of dredges may prove to be the turning point in preventing the river from raising its bed due to other causes than prolongation of the delta.

Mr. van der Veen believes that the soil condition of the river banks has little to do with groin spacing. In the case of very long groins this is true; but for short groins the condition of the soil is an important consideration in spacing. This has been the experience of engineers on the Yellow River.

Mr. Jarvis calls attention to the fact that the Wei Ho is the greatest flood producer and silt carrier of the tributaries and hence requires special attention. In Fig. 1, the writers have shown possible dam sites for detention basins on the upper Wei Ho in Kansu, and also on the King Ho and Pei Lo Ho in Shensi, these latter being feeders of the Wei Ho. Such detention basins may be of great assistance in smoothing out crest flows. It is also in this region that, eventually, extensive soil-erosion control work must be constructed, as it is one of the critical parts of the Yellow River system, especially in the matter of silt flow. The writers have not drawn more attention to the Wei Ho because they believe the main river to be in more immediate need of control. The Wei Ho silt is of such extreme fineness, being mostly of clay composition, that it will not cause the same difficulty along the diked channel as the silt from the main river. Therefore, corrective measures on the drainage area between Tungkuan and Tokoto in Suiyuan should take precedence over similar measures on the Wei Ho drainage area.

The writers had suggested only one auxiliary flood channel—the Tu Hai Ho—or one in its general vicinity. Hills on the southeast bank prevent the location of a similar channel on that side of the river from the old Grand Canal crossing to a point a few miles up stream from Lokou. It is possible, however, to have an auxiliary channel to the south of the river from a point near Lanfeng, just east of Kaifeng, to Tungping Lake and then through the Po Ho to the main river. As Mr. Jarvis suggests, such channels might aid in maintaining capacities in the main river at critical times. In this vicinity there are areas that could take large silt storage in filling depressions.

The writers are highly appreciative of the careful study given this subject by Professor Lane and share with him some of the anxieties he has in the matter of silt transport and disposition. It is agreed that, aside from handling the flood waters and routing them to the sea, there is the closely allied problem of controlling the silt or sediment stream. He is entirely in accord with the writers in emphasizing the need for the further development of laws governing

the transportation of sediment. Professor Lane's discussion is especially valuable since he is able to write not only with a personal knowledge of Chinese river conditions, but also with years of experience in the study of silty rivers in both field and laboratory. Some of his field work has been in the Huai River valley south of the Yellow River.

Professor Lane intimates that a possible danger in a flood-control and silt-control detention basin is that the silt concentration is likely to be a maximum when the outflow from the basin has become nearly that of the natural stream. Hence the Yellow River course through the plain may be in danger of choking. Opinions may differ both in regard to the time of maximum silt concentration in the outflow (see Fig. 20) and the possibility of choking. Since it is not possible to solve these problems by analytical deductions, the writers have recommended that they be solved by laboratory experiments, or, preferably, on small, silty streams such as may be found in Southern California or Arizona.

In this connection actual field data from the Yellow River for August, 1934, show that the diked river is capable of conveying a flood of about 185 000 cu ft per sec, with an 11% silt load, through the entire length of its lower course from Taochengpu to the sea without injuring the channel to any marked degree (see Item 1 in Table 6, Table 7, and Table 4, August 9–20, 1934). Attention is called to the controlling influence of the section between Shanchow and Chinchang on the silt concentration. It is the Mengtsin-Chinchang section of this reach which acts as a temporary storage reservoir when the silt masses become excessive. The next depositing area under present non-regulated conditions is between the 1851 break and Kaotsun near the Shantung border. These two natural storage regions for silt seem, so far, to have prevented the silt concentration in the diked river below Kaotsun from exceeding 12½%. Mr. Freeman records about 10% in 1919 near the Grand Canal crossing. The values for August, 1934, in Table 4, are the highest recorded by the Yellow River Commission for the lower diked river. It seems quite possible, after the channel has become regulated, that discharge of 200 000 cu ft per sec may convey a silt load of nearly 20% without serious deterioration of the channel.

If the Palihutung or Sanmen detention basin project were to be completed and the diked channel regulated, one might expect (see Figs. 19 and 20) that the maximum silt concentration in the outflow would not exceed 40%. (It may even be possible to keep the maximum silt percentage lower than this by gates whereby the stages in the basin could be controlled and excessive silt scour from the basin thus reduced.) Reasoning from what occurs at present along the down-stream channel, one may assume further that the silt concentration in the outflow would be reduced down stream to about 20% by temporary storage between Mengtsin and Chinchang and to 17 or 18% at Kaotsun, from which place the flow would continue with the same silt load right through the diked channel. There might be some deterioration of the diked channel during exceptional floods having very high silt loads; but it seems scarcely reasonable to expect that the diked channel at any place would become choked and the flow sent over the dikes, since in a regulated channel the silt would tend to become uniformly deposited all along the channel. It is also very

probable that the clear outflow from the basin, preceding the scour period, would remove considerable silt from the channel between Chinchang and Kaotsun, the same reach where refill would later take place, thus further reducing chances of choking. Since not all large floods are as heavily silt laden as that of August, 1933 (of which Fig. 20 is an actual record), the channel will again improve in time.

To interpose another silt-depositing reservoir near the up-stream end of the diked course is scarcely advisable although it may be done in case of need. To arrange for such a reservoir would be a formidable technical problem. If, in its simplest form, it is made as a lake, through which the river flowed all the time, depositing most of its silt there, it would become silted up rapidly, even if it had an area of 1 000 sq miles. If it were meant to receive the silty flow only during the periods of heaviest silt concentration, control works would be required both across the main river channel and on the side into the reservoir, as well as for the outlet back to the river channel.

It has been contended several times, in the discussion, that the change in slope along the diked channel will affect, materially, the river's capacity to transport a heavy silt load. This may not necessarily be the case. Field studies made of the Yellow River indicate that slope changes within the limit of those which exist along the diked course are not sufficient to detract seriously from the river's capacity to carry silt. The slope which will begin to affect silt transportation seriously will, most likely, be found to be considerably flatter than those which exist, especially after the channel has been regulated. Of course, there are limits for discharge and silt load beyond which the existing slopes will begin to take effect; but the writers believe that these limits, due to the fineness of the silt, lie much higher than on other rivers—hence, their optimism in regard to the non-choking probabilities in case flood-control and silt-control detention basins should prove feasible.

Under a system of flood-control and silt-detention basins west of the plain, there is the objection that nearly all of the silt will be deposited in the sea and rapidly make the lower diked course untenable. If soil-erosion control in the loess areas is vigorously pursued, however, this objection may not be serious, since the erosion control may be expected to bring imposing results within three or four decades. Nevertheless, the possibility exists that such control may not be sufficiently effective, and the problem arises of how else to handle the silt. Professor Lane pointedly draws attention to this and suggests three methods:

1. Keeping it in place on the water-shed (may not be very effective);
2. Taking part out of the river channel and storing it on the plain; and
3. Carrying a part out to sea and depositing it on the sea floor.

The writers concur fully with Professor Lane in these suggestions, but at the same time they maintain that it is of paramount importance to regulate the channel, as without regulation it will be impossible to enforce any other control. Nothing can be done with a river which is likely to break loose anywhere and which wanders at will over a broad strip of land within irregular lines of dikes spaced excessively far apart. The silt problem must be handled quan-

titatively, and this can be done first after the channel has been regulated. It must be determined how much silt goes to the land, how much is left between the dikes, and how much goes to the sea. These quantities should be so proportioned that they will give the longest life to the entire diked channel. To obtain the desired uniformity and quantitative control of deposition in channel, outside areas, and sea, it is essential that the entire channel be so regulated that there is no chance for any outflanking of the control works. The outflanking of the Yung Ting Ho diversion works in 1933 and 1934 was due to the fact that the river up stream was unregulated. This and other illustrations of costly gambling with silty rivers in North China remind engineers of the prudence of stabilizing and regulating these channels.

It has been suggested that the writers might have considered the matter of silt disposal more fully in their recommendation for control measures. To do so would have meant, evidently, a quantitative appraisal of silt disposal. Data for doing so, unfortunately, are still insufficient. Professor Lane suggests certain principles along which such silt disposal should be guided, but at the same time recommends laboratory research for quantitative determinations. To this the writers agree, and here the matter must rest for the time. With China at war there seems little opportunity of this research work being undertaken in that country. Possibly a way will be found to solve some of these problems in the United States so that a comprehensive control of the Yellow River will be planned with greater assurance that the element of chance has been removed to a very large degree.

The proposals of Mr. Higgins have been answered in replies to previous discussions. The writers believe that they have remarked fully enough on the possibility of letting the river "handle its own problem." They are convinced that more control, rather than less, is essential, especially where there is such a dense population to be considered. Resourceful though they be, the Chinese farmers need direction in matters such as control of Yellow River floods, because these come as great avalanches of water surcharged with mud. Too long, the people of China have been victims of these river "dragons." It is time for the entire engineering profession to give the very highest service in its power to bring this river under control. It is a task for hydraulic engineers with years of intimate river experience.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

THE RISK OF THE UNEXPECTED IN SUBSURFACE CONSTRUCTION CONTRACTS

Discussion

BY OREN CLIVE HERWITZ, ESQ.

OREN CLIVE HERWITZ,[16] ESQ. (by letter).[16a]—Mr. Newton's discussion of the cases cited and analyzed in the paper was very helpful, for he refers in several instances to additional facts which the writer, because of space limitations, found it necessary to omit; but, with several of Mr. Newton's observations the writer is constrained to disagree.

Thus, Mr. Newton's statement that "an owner who has given the contractor inaccurate or incomplete boring information is responsible even if the boring information is not made part of the contract" is misleading and incorrect, for the Court of Appeals of the State of New York has held that in such a situation the owner is responsible only if guilty of bad faith. The Court said (42):[16b]

"If, however, notwithstanding the agreement as to honest mistake, damages might be recovered from the State for misrepresentations, upon which the bidder might rely, the boring sheet was not such a representation. It formed no part of the plans upon which the contract was based. It was not prepared or used for that purpose. It was an independent bit of information or supposed information in the possession of the State to which the bidder resorted in making the investigations which it was required to make. If it relied upon this paper, it did so at its own risk. The most it could ask for in regard to this information was good faith."

Mr. Newton's further statement that "a contractor is not required to make surveys, soundings, or borings to determine whether the information given him in the contract is correct" is equally misleading, especially when compared with his subsequent remark that "A bidder is not usually required to make borings

NOTE.—This paper by Oren Clive Herwitz, Esq., was published in January, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1939, by Messrs. Alonzo J. Hammond, Frederick W. Newton, David A. Molitor, F. B. Marsh, and Evan S. Martin; May, 1939, by Messrs. C. Maxwell Stanley, and Francis J. Morgan and Frederick C. Zeigler; June, 1939, by Messrs. Lazarus White, and T. Kennard Thomson; and September, 1939, by Carroll A. Farwell, M. Am. Soc. C. E.

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[16a] Received by the Secretary November 16, 1939.

[16b] For reference to numerals in parentheses, see "Court Citations and References," in the Appendix of the paper and at the end of this discussion.

or other sub-surface investigation, particularly if the work has been done by the owner." Mr. Newton's first statement is too broad, for the Court of Appeals of the State of New York has indicated that situations may arise where the contractor might be held responsible for what borings or soundings made by him would have disclosed. The Court said (60):

"He [the bidder] had the opportunity to physically examine the location of the work to be done, and if that called for extensive tests, by way of borings and soundings, that fact, however troublesome, was one he must take into consideration. He had presented to him an agreement, which was precise and strict in its requirements, and which cast upon him the possible risk that all might not be as it appeared, or as estimated by the surveyor. Admittedly, borings and soundings would have revealed the imperfect performance of the previous grading contract, in the leaving of native rock under the surface of the roadway."

Finally, Mr. Newton's discussion of the cases, in which he makes almost constant reference to the wording of particular contract clauses, is inconsistent with his earlier statement that "too much emphasis" can be placed on the wording of the contract, and that "the facts, not the contract clauses, form the real basis" of the Court's decision. It is the writer's belief that the language in the contract is of primary importance, for that is all that the Court may consider in determining whether owner or contractor was, by their agreement, intended for the rôle of risk-bearer. That intent, once determined, is controlling, and the Court is powerless to do anything but enforce it. As the Court said in Johnson vs. City of New York (83):

"In the absence of fraud or mistake, the contract made by the parties, however burdensome it may prove in execution, must govern. It is no excuse that a party to a contract did not foresee a hardship which its operation might entail. It is the function of courts to enforce contracts according to their true intent and meaning and neither to mitigate nor relieve against their operation."

The proposition that Courts of law, in deciding contract disputes, are doing nothing more than enforcing the intent of the parties, as expressed in their agreement, is one which laymen find difficult to comprehend. Judges cannot make a new contract for the parties; nor can they rewrite a contract so as to make it fairer to one party or the other. If (as so many of those taking part in the discussion seem to feel) it is unfair and inequitable to make the contractor assume the risk of unexpected sub-surface conditions, Court decisions which have that result are not the fault of judges but rather of the parties who willingly signed a contract containing language to that effect.

Mr. Martin states that the judicial opinions cited in the paper "are not much concerned with what is fair but, as a lawyer might say, what is fair 'in the light of the contract.'" He is quite right, if by that he means that the Courts cannot decide contract disputes upon general principles of ethics or morality. On the contrary, the Court's "hands are tied down" to the standard of ethics and morality previously fixed and determined by the parties themselves in the contract which they have signed. Although it is true that, in many cases, a contractor is compelled by the Courts to bear the loss resulting from an unexpected sub-surface condition, it is only because he assumed that risk in his

contract; and the Courts, much as they might like to, are powerless to relieve him of the consequences of his unprofitable bargain.

When owners, contractors, and engineers realize that the solution of their difficulties lies not in catch-all contract clauses, but in a careful choice of unambiguous language to express their contractual obligations, there will be no occasion to criticize Courts for decisions which simply place the responsibility on the party who agreed to assume it.

Of course, the question as to whether or not it is sound, ethically or economically, for the contractor to assume the risk of unexpected sub-surface conditions (with which the discussion was almost exclusively concerned) is one which belongs more properly within the sphere of the engineer rather than that of the lawyer. There is much force in the arguments of Messrs. Hammond, Molitor, Marsh, Stanley, White, Thompson, and Farwell against making the contractor assume all the risks. On the other hand, there may be circumstances, as Mr. Farwell points out, under which the owner may feel compelled to require the contractor to do so, even if he thereby pays a higher upset price for the completed structure in order to cover the contractor's contingencies.

It is the writer's thesis that, whatever may be the decision reached in any particular case, the contract should express that decision, clearly and fairly, and in no uncertain terms. If a contractor is expected by the owner to assume all risks, he should be definitely advised of that fact so that appropriate allowances can be made in his bid. If he is not to bear the risk, it is to the owner's advantage that this also be made clear, for then the bids received will not be inflated by the contractors' fear of the unknown. In either event, neither party can have cause to complain or to litigate, for each will know the nature and extent of his contract obligations.

However, this ideal contract, in which there is a "clear meeting of the minds," cannot be wished into existence. The preparation of such a contract requires care and precision in the choice of language, a familiarity with the rules of contract interpretation customarily applied by the Courts, and a working knowledge of the fate that typical contract clauses have suffered in previous legal battles.

Realizing that in most instances construction contracts are drawn by engineers and not by lawyers, the writer hoped that his brief summary of the case law on this subject might prove helpful. If the necessity for careful, frank, and forthright contract draftsmanship has been emphasized, and the means to achieve it suggested, the paper may have served some useful purpose.

Court Citations and References.—

(83) 191 App. Div. 205, 181 N. Y. Supp. 137, aff'd 231 N. Y. 564, 132 N.E. 890.

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DISCUSSIONS

EARTHQUAKES AND STRUCTURES

Discussion

BY JOHN D. GALLOWAY, M. AM. SOC. C. E., AND LYDIK S. JACOBSEN, ESQ.

JOHN D. GALLOWAY,[35] M. AM. SOC. C. E., AND LYDIK S. JACOBSEN,[36] ESQ. (by letter).[36a]—Some points made by the several writers who have discussed this paper seem to require further explanation in order to make the subjects more clear.

Mr. Hadley seems to doubt the value of the theoretical treatment of design of structures to resist earthquake waves, owing to the great complexity of the forces. The value of the theoretical treatment lies in the fact that, if the treatment is correct, there will result a consistent design of the structure. The assumptions are necessarily uncertain and should be simple. However, when once made, there is a possibility that the design will be so consistent that no part of the structure will be found wanting when subjected to forces of the shock. It is to be doubted whether Mr. Hadley would advocate designing a structure where "good guesses" formed the basis of design. Under such a procedure, some brace or other essential member might be left out, and the whole structure brought to failure thereby. Experience, such as that indicated by the writers, is a guide in making the assumptions upon which theory is based, such as the acceleration expressed as a percentage of gravity, which is of the greatest value. Once this is determined, theory may be brought in to produce a fairly consistent design. The writers must differ from Mr. Hadley in his statement that "* * * it is not apparent that the weight of the construction has any appreciable bearing on the matter at issue." The mass of

NOTE.—This paper by Leander M. Hoskins, Esq., and John D. Galloway, M. Am. Soc. C. E., was published in December, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1939, by Messrs. Homer M. Hadley, and R. McC. Beanfield; April, 1939, by Messrs. R. S. Chew, Jacob J. Creskoff, and Arthur C. Ruge; May, 1939, by Walter L. Huber, M. Am. Soc. C. E.; June, 1939, by A. A. Eremin, Assoc. M. Am. Soc. C. E.; and September, 1939, by Henry D. Dewell, M. Am. Soc. C. E.

[35] Cons. Engr., San Francisco, Calif.

[36] Prof. of Mechanical Eng., Stanford Univ., Stanford University, Calif. [Prior to his death on September 8, 1937, Professor Hoskins had persuaded Professor Jacobsen to act for him in preparing the closing comments of this paper. Throughout, the word "writers" is interpreted to mean Messrs. Hoskins and Galloway, with the understanding that, in the closing discussion, Professor Jacobsen is interpreting his knowledge of Professor Hoskins' views.]

[36a] Received by the Secretary October 31, 1939.

the structure measures the forces set up therein by the earthquake. Reduction of mass, with the corresponding decrease in stresses, permits a design in which such stresses can be taken care of in a better manner. This remark applies particularly to the joints of an articulated structure. In regard to visible surface waves in earthquakes, it is suggested that Mr. Hadley examine reports of various earthquakes where such waves have been reported. Possibly the experience of one excited observer in Tokyo and a quotation from Shakespeare would not be considered sufficient to offset the uniform testimony of the existence of such waves.

The comments by Mr. Beanfield constitute an interesting summary of the problem from his point of view. The writers are in agreement with most of his remarks, but they cannot concur with his positive statement in Item (6) that "It is a well-established fact that the only proper method of judging earthquake intensity is by the kinetic energy method." It is true that much is in favor of this method, and that rationally it has a good foundation, but it has not yet become "a well established fact." It remains an hypothetical, although a plausible, method.

Suggestion (*a*) offered by Mr. Beanfield is a rule for computing the "maximum period, acceleration, and dynamical shears," presumably on the criterion of maximum fundamental mode deflection within the elastic limits of the materials. However, the statement that "the maximum statical deflection * * * [due to 0.1 g] is also approximately the maximum amplitude of its transient vibrations" seems to be without theoretical backing; at least the reference given to Timoshenko's book[11] does not warrant such sweeping conclusions. Again, under Suggestion (*c*) the statement that "The natural period of a structure ascertained by the foregoing method should not exceed 0.5 sec" is blunt and arbitrary.

The writers agree with Mr. Chew in advocating that empirical formulas for fundamental periods of buildings be used whenever sufficient observational data are available for existing buildings in the vicinity.

The influence of the non-rigid ground on the periods of a structure is largely a matter of guess work even if, as Mr. Creskoff suggests, dominant periods of the site had been obtained before the building was erected. For this reason, it is questionable what is meant by "reasonable accuracy" in the determination of the period of a projected structure. The usefulness of Table 2, given by Mr. Creskoff, would depend on the designer's ability to estimate the period ratios $\frac{T_b}{T_g}$, and even then the use of Mr. Creskoff's table is extremely arbitrary.

The discussion by Professor Ruge mentions the fact that a closer "tuning in" of earthquake frequency with structural frequency is necessary for the higher modes of vibration if equal dynamic magnification is to be expected. The writers agree with this fact and admit that the absolutely "steady state" of vibration, considered in the paper, is so idealized that application to actual conditions must be tempered with considerable insight and experience on the part of the designer. Professor Ruge also calls attention to the fact that a

[11] "Vibrational Problems in Engineering," by S. Timoshenko, Rayleigh's Energy Method, p. 63.

large building will interact with, and influence, the ground on which it stands to such an extent that the forces of the moving ground are not unlimited. This is also in agreement with the view of the writers; moreover, this interactive play of a large building with its foundation makes the period ratio $\frac{T_b}{T_g}$, used by Mr. Creskoff as well as by the writers, rather indeterminate.

Mr. Huber differs from the analysis of the writers which holds only for the "steady state" and in his investigations of the Seal Beach Power Station chimney he used the "impact theory" as developed by Professor Le Conte. It will be recalled that the "impact theory" is based upon the supposition of infinite rigidity of a perfectly free rod—that is, a rod without elastic distortions and consequently without vibrational properties. One must admit that actual structures fall short of approximating these assumptions. In the case of the Seal Beach chimney there was an agreement between the observed damage and the results of the theory. However, the location of the damage is equally well explained by the elasto-dynamic theory if it is supposed that the second mode vibration was excited instead of the fundamental mode. Calculations and observations make it seem highly probable that the earthquake of March 10, 1933, excited the second mode of the chimney much more violently than the fundamental.

The analysis of this particular chimney indicates the difficulties that confront an engineer when called upon to determine what caused the damage to the structure during the earthquake. Here was the case of a very high and heavy chimney held aloft upon a steel frame—a very unusual design, but one not entirely unknown. Mr. Huber adopted an analysis based upon a certain theory and the observed effects of the earthquake seemed to correspond thereto. Another theory probably would have reached substantially the same result and also might have justified the recommendation made by the engineer that the chimney be reduced in height. No mistake was made in so doing, but the writers are of the opinion that the weight of evidence favors the elasto-dynamic theory.

In this connection attention may be called to the fact that the speed of the earthquake waves is such that when any structure is encountered, the effect would be similar to a blow or sudden impact. The waves move at a speed of several miles per second and it is a well-known fact, as in bridges, the suddenness of the blow such as from a moving train is of material effect in increasing the stresses in the structure. In modern bridge design, impact stresses play a material part in the design. The entire discussion indicates the complexity of the subject and demonstrates how differences of opinion may arise.

The writers agree with Mr. Dewell's suggestion, namely, to amplify the statement under the heading "Earthquakes and Their Effects: Relation to Structures." The analysis given by Mr. Dewell, of a 220-ft chimney, is also very interesting. Again, the moments and shears due to the existence of higher mode vibrations will explain Mr. Dewell's quoted statement, due to I. Hiroi of Tokyo, that the damage to a chimney might be located "almost anywhere in the

whole height."[34] The writers concur with Mr. Dewell in believing it more probable that earthquake fractures of free-standing chimneys are due to the effects of vibration rather than of impact.

The substance of the writers' paper lies in the statements of the complexity of the problem and the necessity for some simple assumptions that would make the problem one in which a consistent mathematical treatment was possible. There does not seem to be any material differences among those who have discussed the paper, as to the treatment adopted. It may be assumed that there is a general approval of the assumptions made by the writers. It is not contended that the treatment represents the last word on the subject, but rather the reverse is true. As more data are accumulated upon the subjects of earthquakes—especially as to the duration of the movement, the acceleration, the characteristics of the waves and (which is important) the similarity to assumptions that are within the range of mathematical treatment—a revised treatment may be possible that will accord with the additional knowledge acquired.

[34] "Prevention of Damage to Engineering Structures Caused by Great Earthquakes," by I. Hiroi, Paper No. 117, *Proceedings*, World Eng. Cong., Tokyo, 1920, Vol. 9, p. 25.

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DISCUSSIONS

THEORY OF LIMIT DESIGN

Discussion

BY A. A. EREMIN, ASSOC. M. AM. SOC. C. E.

A. A. EREMIN,[55] ASSOC. M. AM. SOC. C. E. (by letter).[55a]—In the last half of the Nineteenth Century various theories of failures in structural materials were developed. However an extensive theoretical and experimental study is needed to apply them to the computation of the stability of structures. Development of the limit-design method based on the theory of failure may be found in various practical rules applied to the design of structures.

The most common application of the limit-design method is in the design of foundations for statically indeterminate structures. It is well known that foundation deformations do not follow Hooke's law. Therefore, in each case the designer establishes the maximum possible displacement of the footings, and computes the factor of stability of the structures in carrying a given loading. An interesting illustration of the practical application of limit-design computation is in determination of the critical loading carried by a group of piles under spread footings. The piles with the least bearing resistance settle faster and approach a uniform distribution of loading on all piles in the group, which determines the limit-design loading. Computation of the critical loading carried by structural details, such as hinges or pins, is often based on practical rules established by the principles of the limit-design method. Likewise, in the design of rivet connections the principles of the limit design may be considered. Interesting computations of the distribution of deformations in rivets were shown by the late G. F. Swain, Past-President and Hon. M. Am. Soc. C. E.[56] An application of the limit-design method to the computation of the critical loading in statically indeterminate structures has been demonstrated by Professor

NOTE.—This paper by J. A. Van den Broek, M. Am. Soc. C. E., was published in February, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1939, by Messrs. John H. Meursinge, I. K. Silverman, Edward Godfrey, Basil Sourochnikoff, E. Mirabelli, C. M. Goodrich, George Winter, and Francis E. Simpson; June, 1939, by Messrs. Joseph A. Wise, Alfred M. Freudenthal, Hans H. Bleich, Alfred S. Niles, and A. Floris; September, 1939, by Messrs. L. H. Nishkian, and F. G. Eric Peterson; and October, 1939, by E. S. Fabian, Assoc. M. Am. Soc. C. E.

[55] Associate Bridge Engr., Bridge Dept., Div. of Highways, State Dept. of Public Works, Sacramento, Calif.

[55a] Received by the Secretary November 8, 1939.

[56] "Structural Engineering: Strength of Materials," by G. F. Swain, 1924, McGraw-Hill Book Co., Inc., p. 408.

Timoshenko.[57] Professor Timoshenko has shown that strains in a member with stresses above the elastic limit may be expressed by the average modulus of elasticity, and a critical loading carried by the structure may be computed with the equations based on the elastic theory.

It is unfortunate that the author, in Cases 1 and 3, has considered the beams with constant moments of inertia. In important srtuctures the moment of inertia is varied according to the maximum bending moment stresses. Evidently the economic relation of the limit-design and elastic methods in the design of beams with curved haunches will not be the same as that shown by the author.

The elastic theory is advantageous in that it may readily be verified by tests, whereas the limit-design method will require greater experimental and theoretical study before conclusive results will be obtained.

[57] "Strength of Materials," by S. Timoshenko, Part II, p. 668.

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DISCUSSIONS

POLLUTION OF BOSTON HARBOR

Discussion

BY MESSRS. F. E. DEMARTINI, AND A. M. RAWN

F. E. DEMARTINI,[8] ASSOC. M. AM. SOC. C. E. (by letter).[8a]—In many respects this excellent paper could apply as a description of pollution in San Francisco Bay, in California. It may be of interest to outline briefly the situation there.

Until February, 1939, the untreated sewage of San Francisco was discharged to San Francisco Bay through twenty-eight outlets. Approximately half of these outlets discharged sanitary sewage and the others discharged storm water and varying quantities of sanitary sewage, depending upon the extent and intensity of rains. Three of the outlets—North Point, Bakers Beach, and Mile Rock—discharged the sanitary sewage from the largest proportion of the city's population. Local nuisances at some of the smaller outlets are critical, particularly in the ship channel parallel to Channel Street, and in Islais Creek. Sludge deposits have been extensive here in the past, resulting in extensive odors at low tide.

In 1932 a 1.0 mgd activated sludge sewage treatment plant was constructed by the Park Department, in Golden Gate Park, to reclaim sewage for irrigation of park lands and sludge for use as fertilizer. This was the first treatment plant of any importance built in the city.

In February, 1939, the 15.0 mgd Richmond-Sunset Sewage Treatment Plant started operation. At times of heavy rains and diluted sewage the plant is designed to handle peak flows of 55.0 mgd. This is a primary treatment plant for removal of grit, grease, and sludge. Chlorinating equipment is provided for the control of odors and disinfection of plant effluent. Briefly the factors leading to the construction of this first major treatment plant were as follows:

Disposal of raw sewage direct to the Bay waters resulted in (1) heavy pollution of a number of bathing beaches, (2) unsightly sleek areas, and (3) floating sewage solids and odors at many points along the bulkhead line on the

NOTE.—This paper by Arthur D. Weston and Gail P. Edwards, Members, Am. Soc. C. E., was published in March, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1939, by E. Sherman Chase, M. Am. Soc. C. E.

[8] Associate Public Health Engr., U. S. Public Health Service, Cincinnati, Ohio.

[8a] Received by the Secretary November 6, 1939.

water front. Practically all areas from which clams and oysters had been taken on the San Francisco side of the Bay were closed by the California State Department of Public Health in recent years due to increasing pollution of the Bay waters.

The complacent attitude of the city toward this pollution of San Francisco Bay underwent a gradual change starting about 1931. Investigations by the city and State health departments were the principal stimuli to the new viewpoint. These investigations definitely indicated the gross pollution at Bakers Beach, Aquatic Park, and Yacht Harbor in the Marina. These recreational areas are used extensively by large numbers of people.

Conditions at China Beach were studied by the California State Department of Public Health in connection with a project to make the beach and adjacent lands a State and city park. Shortly thereafter a bacteriological study of the bay waters along the northerly and westerly shore line of the city (from pier 45 to the Funston Life Guard Station) was undertaken for one year by the San Francisco Department of Public Health. Both of these investigations indicated the serious extent of the pollution and the magnitude of the problem involved in its elimination. By 1934 interest had been aroused to such an extent that a board of consulting sanitary engineers was appointed by the city to study the problem of treatment and disposal of sewage from the city and county of San Francisco. In May, 1935, this board submitted a report and recommendations to the city.

A comprehensive plan was presented providing for the concentration of the city's sewage at a few points and for treatment of sewage before disposal into San Francisco Bay and the Pacific Ocean. In accordance with this plan sewage from the Marina District has been pumped to the North Point Outlet since 1937, and the Richmond Sunset Sewage Treatment Plant was completed in February, 1939. The sewage from Bakers Beach District, Richmond District, and Sunset District is treated at the new Richmond-Sunset Sewage Treatment Plant, prior to disposal into the Pacific Ocean at the Mile Rock Outlet. These developments will greatly improve conditions at a large number of small beaches and particularly at the Marina, Bakers Beach, and China Beach.

In addition to San Francisco there are a large number of communities discharging raw sewage to San Francisco Bay. A few places, among which are Burlingame, San Mateo, Palo Alto, and Petaluma, have built treatment plants, but there remain a large number of places that are seriously polluting the bay waters. There is no doubt that the program undertaken by the city of San Francisco, involving the expenditure of several million dollars, will have a salutary effect on the attitude of these communities toward elimination of their pollution load from the Bay.

Many details of the studies reported by the authors for Boston Harbor and the general recommendations for treatment and improvements are strikingly similar to those of the San Francisco Bay area.

Corrections for *Transactions:* In *Proceedings* for March, 1939, page 384, line 13, change "76 086 gal" to "76 086 million gal"; page 388, line 31, delete "North"; page 390, lines 13 and 14, delete the clause "which extends from Deer

Island Pumping Station to the vicinity of Deer Island Light." Page 395, line 22, change "ground" to "water"; and, in the caption of Fig. 8, change "Ammonia" to "Free Ammonia."

A. M. RAWN,[9] M. AM. SOC. C. E. (by letter).[9a]—The authors' description of the investigation of the pollution of Boston Harbor and of corrective measures proposed raises a point or two in the writer's mind which, in connection with a problem of this nature, may merit some discussion.

It is not stated in the narrative whether the tubes from which data were gathered for preparation of the "isocols" in Fig. 9 were set immediately after the samples of mixed sewage and sea water were taken, or were iced and set later. The procedure here may have an important bearing upon conclusions because of the fact that sea-water plankton destroy bacteria quite rapidly, and time becomes an important factor in arriving at the number of tubes positive. This is well illustrated in Fig. 12. If the samples were set immediately after they were taken, the "isocols" shown in Fig. 9 may be said to represent a fairly true degree of pollution. However, if they were iced and set some hours later, a situation more aggravated than that shown in Fig. 9 might exist.

As an alternative to discharging sewage at the Graves, the authors propose plain subsidence works for the Boston Main and the North and South Metropolitan Sewerage Systems. It is not clear to the writer that construction of plain subsidence works at the points indicated by the authors, followed by the discharge of settling-tank effluent at present outfalls, will have any great effect upon the spread of contamination over the surface of Boston Harbor waters. In all probability, it will prevent the washing of objectionable material identified as of sewage origin on to the beaches; but the sewage discharged at or near the surface or through the sea outfalls, at present locations, will probably spread over and pollute as great an area within the harbor as is now the case with discharge of raw sewage. The fact that the harbor mud is polluted to the extent indicated in the authors' description seems to testify that a large proportion of the suspended material in the sewage settles quite promptly in the comparatively still harbor waters, as is to be expected. Such being the case, surface pollution, even at the present time, must be due largely to the spread of mixed sewage and sea water from which much of the suspended matter has settled. The writer's criticism is entirely invalid if plans contemplate the sterilization of the effluent prior to final disposal.

[9] Asst. Chf. Engr., Los Angeles County Sanitation Dists., Los Angeles, Calif.

[9a] Received by the Secretary November 17, 1939.

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DISCUSSIONS

DESIGN OF CIRCULAR CONCRETE TANKS

Discussion

BY GEORGE S. SALTER, M. AM. SOC. C. E.

GEORGE S. SALTER,[45] M. AM. SOC. C. E. (by letter).[45a]—It has often been noted that one of the chief values of a paper is that it brings out some interesting and instructive discussions. Several such discussions were presented on this paper.

Messrs. Silverman, McCormick, Moorman, and Young each brought out clearly, and correctly, that the solution of Equation (10) is nothing new. In this regard the writer wishes to stress the fact that the paper was conceived in the studies made on the design of a particular type of tank—settling tanks for sewage treatment which were 126 ft in diameter with water depth of only 12 to 13 ft. A study of the English literature on the subject gave very meager information on design for tanks of this proportion as, in most cases in the solution of Equation (10), the value of the arbitrary constants A and B had been assumed as equal to zero. Preliminary studies indicated that such an assumption led to results considerably in error and thus the primary purpose of the studies was to evaluate these constants for the conditions as stated in the paper. It was found that the values of these constants were of considerable importance for tanks in the lower range of values of $\frac{H^2}{r\,t}$. This is indicated clearly in Fig. 2. Since the publication of the paper it has been brought to the writer's attention that apparently a considerable part of the work had been done previously and that various graphs had been presented, chiefly in German publications, some of which are practically the same as some of those given in the paper.

Mr. Rosengarten suggests the use of pre-stressed rings at the top of the wall in order to simplify the design. This method, and also the method of using a pre-stressed band for the entire height of the wall, were studied; but

NOTE.—This paper by George S. Salter, M. Am. Soc. C. E., was published in March, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1939, by Messrs. Carl A. Rosengarten, and I. K. Silverman; and June, 1939, by Messrs. Frank J. McCormick, H. B. Muckleston, Robert B. B. Moorman, Dana Young, L. J. Mensch, Jens Egede Nielsen, Lloyd S. Dysland, Basil Sourochnikoff, and Regis F. Fey.

[45] Subway Structural Designer, Subways and Traction, Chicago, Ill.

[45a] Received by the Secretary November 14, 1939.

it was found that they were patented features and also of doubtful savings for the conditions that prevailed.

Messrs. Muckleston, McCormick, Silverman, and Young have stated that the method is not strictly exact in that the value of Poisson's ratio has been neglected. The writer neglected this factor intentionally and made other assumptions as mentioned by Messrs. Muckleston and Young in order to simplify the mathematical work. It was not believed that any of these assumptions were of great importance. The word "exact" was used by the writer in contradistinction to the approximate methods used when the arbitrary constants A and B were assumed equal to zero. Mr. McCormick goes on to show that for all practical purposes the assumptions made, although not exact, may be justified.

The writer agrees with Professor Moorman that Equation (17) would be understood more clearly if written as he suggests in Equation (30). Equations (32) and (33) are more exact and should be used instead of Equations (18) and (19). Professor Moorman also shows that there is a close agreement between the results secured by the writer's method and that given by W. S. Gray. They should agree as the basis for the two methods appears to be the same.

Mr. Young reveals, clearly, the importance of considering how closely the actual conditions conform to those assumed in the analysis. Probably the most important of these conditions is the restraint at the base of the wall. If the base of the wall is not fixed the values of the moments, shears, and ring tension will be changed materially from those obtained under conditions of fixed base. He also mentions that the analysis may be simplified considerably when the value of θ is greater than π. This simplified analysis has been presented earlier by Smith, Rasmussen, Leeper, and others. The writer, as stated before, was interested primarily in tanks of proportions whose θ-values were close to, or less than, π.

Mr. Dysland's data on estimated savings for the fixed-base tank over the same tank designed for hoop tension only are similar to some studies made by the writer. As the studies made were preparatory to the design of two batteries of sixteen tanks each it will be seen that the total savings would be considerable if Mr. Dysland's data were even approached.

Previous to the plotting of Fig. 3 the writer made up diagrams showing the moment at any height of the wall for the same θ-values as given in Fig. 4. From these diagrams the point of inflection is found readily.

It was not intended to infer that the method could be used for either internal or external loading to any partial height of the wall. The conditions assumed in finding the values of the arbitrary constants state that the moment and shear at height H are zero. This condition obtains only when the loading extends to, or reasonably close to, the top of the wall.

Mr. Mensch states the problem and its answer, correctly, in that American literature contained very little on the subject of design of large-diameter shallow tanks and that the writer's method was developed to design a tank of these particular proportions. A difference of opinion appears to exist as to how much confidence designers will have in such theories. It was hoped that

if the method met with the general approval of those who did follow the analysis the method would be of some value in the relatively limited field to which it particularly applies. It might be added that thirty-two tanks, 126 ft in diameter, with a water depth of 12.5 ft, and eight tanks, 75 ft in diameter, and water depth of 14 ft, were so designed and built and are now (1939) in successful operation.

In conclusion the writer wishes to thank all who have added to the value of the paper by their discussions.

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DISCUSSIONS

FLASH-BOARD PINS

Discussion

BY JOHN E. FIELD, M. AM. SOC. C. E.

JOHN E. FIELD,[11] M. AM. SOC. C. E. (by letter).[11a]—In pointing out the uses and desirability of the pipe-supported, collapsible flash-board, this paper is so complete that little remains to be added. The authors are to be commended for the great care with which their research was conducted, and to be congratulated for the very practical conclusions made available to the engineering profession. The writer has used, and caused to be installed, such flash-boards in the spillways of canals and reservoirs, but until he was furnished with advance information and data by the authors, he has hesitated in recommending them unless installed under his personal supervision.

Spillway costs are no inconsiderable part of the entire cost of a dam, and with "fool-proof" collapsible flash-boards that cost is reduced materially and much of the valid objection to flash-boards in spillways will be answered satisfactorily.

The writer has seen many, and has installed some, collapsible flash-boards dependent more or less on mechanical operation, and his remembrance is that few were successful and reliable.

If the dam and spillway are close to a power-plant where experienced men are available at all hours, and especially where there are telephones to give warning of approaching floods, mechanically operated flash-boards in spillways can be tolerated.

The publication of the paper is opportune as records of the size of maximum floods have been considerably changed in the last few years; and ideas of what constitutes necessary spillway capacity have been greatly altered, with the result that the capacity and cost of spillways have been increased materially. Many existing spillways, heretofore considered to be of ample capacity, must be increased in size to conform to the probable maximum flood as determined by the later data.

NOTE.—This paper by Chilton A. Wright and Clifford A. Betts, Members, Am. Soc. C. E., was published in May, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1939, by Messrs. William P. Creager, Lincoln W. Ryder, and E. T. Schuleen.

[11] Cons. Engr., Denver, Colo.; Engr. Examiner, PWA, Washington, D. C.

[11a] Received by the Secretary November 6, 1939.

The failures of more than 100 dams in New England, New York, and New Jersey, in March, 1936, were due to inadequate spillways and are a forceful argument for the re-examination and change in the design of existing spillways. An increased depth by lowering the floor or lip of the spillway with collapsible pipe-supported flash-boards, installed to maintain the necessary height of water in the reservoir, will often be the answer to the new problem.

Two valid objections to an uncontrolled spillway are the probable loss of stored water, and the difficulty of stopping the outflow after the danger point has passed. These objections can be met in part by so selecting the pipe supports that those most accessible and easily replaced will fail before others. The authors have studied a flash-board so designed that the upper part collapses at a predetermined height of water and the lower part at a higher stage of water level. For a narrow spillway, pipe set horizontally in sockets, in both of the concrete abutments, could be made to support an upper portion of the flash-board independently of the lower portion, and would be free to swing down stream on hinges at the abutments. Local conditions will suggest many variations in the details of the design all based on the fundamentals shown in the paper. The writer has found this type of flash-board very satisfactory in canals, and the description of one actual problem may be of interest.

The cross drainage was taken into the canal primarily to secure the water for storage and also to obviate an expensive culvert or overshot. The cross drainage water from moderate rains could be carried by the canal; but storms were quite common during which the storm water would be in excess of the capacity of the canal to carry it. The proper location for the spillway was at a rock cut, up stream from the cross drainage channel; the lower bank of the canal was cut down to a level 2 ft below the high-water line and pipe supported flash-boards installed in the cut. They were designed to collapse when the water rose 1 ft above the high-water line so that when the pipe failed the waste-way would carry a depth of 3 ft of water equal to the maximum estimated inflow from the cross drainage. Under certain conditions the flood water will flow up the canal and be discharged through the spillway; this prevents an excessive flow below the point at which other cross drainage is probably entering the canal. The plans called for three such wasteways although the canal was provided with a regular wasteway with its floor somewhat lower than the bottom of the canal. This wasteway was provided with slide gates and hand-operated lifting devices.

In 1905, before the pipe-supported flash-boards were proposed, the writer provided wasteways in one large canal by cutting the lower bank, but in lieu of collapsible flash-boards a low earth dam was placed in the spillway. This dam had a crest level 1 ft above the high-water line and washed out whenever overtopped, thereby relieving the canal of its excess of cross drainage water. This design was considered "fool-proof," certain in operation, and easily replaced. The writer is now wondering why he did not think of pipe-supported flash-boards in this case; and he would have profited greatly had the contents of the paper been available to him at that time.

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DISCUSSIONS

COMBINING GEODETIC SURVEY METHODS WITH CADASTRAL SURVEYS

Discussion

BY WILLIAM L. SAWYER, ASSOC. M. AM. SOC. C. E.

WILLIAM L. SAWYER,[7] ASSOC. M. AM. SOC. C. E. (by letter).[7a]—Mr. Berry's paper is an excellent description of an extended survey, including geodetic and cadastral methods, which, until recently, were rarely associated. From the manner in which these were correlated, it is evident that careful planning was a necessary preliminary to undertaking the work. This paper may well serve as a reference for any Federal, State, or political subdivision which is undertaking an extended survey, especially where cadastral surveys are involved.

The economy claimed by the author in the execution of the work is not to be disputed. In this connection it is gratifying to note that the cadastral points were defined in terms of plane rectangular co-ordinates on the Lambert conformal projection, thus tying the cadastral surveys to a system by which lost corners may be easily recovered or replaced with precision. Thus the economy to be realized by the many advantages of plane rectangular co-ordinates will be extended into the future. Philip Kissam, Assoc. M. Am. Soc. C. E., has discussed these points in an excellent manner.[8]

The writer doubted the advisability of using 7-in., 0° 0′ 10″ repeating theodolites as direction instruments on second-order triangulation until noting the results of triangle closures in Fig. 8. The requirements of the U. S. Coast and Geodetic Survey[6] for second-order triangulation angles are, for average triangle closure, 0° 0′ 3″, except that an effort should be made to keep the average closure down to 0° 0′ 2″.5, and that the maximum closure should seldom exceed 0° 0′ 6″. With one exception (arc Kettle 3″.38) the triangle closure is less than 2″.5. The maximum triangle closure ranges from 4″.45

NOTE.—This paper by Carl M. Berry, Assoc. M. Am. Soc. C. E., was published in September, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[7] Associate Prof., Civ. Eng., Coll. of Eng., Univ. of Florida, Gainesville, Fla.

[7a] Received by the Secretary November 10, 1939.

[8] "Proposed Improvements for Land Surveys and Title Transfers," by Philip Kissam, *Proceedings*, Am. Soc. C. E., April, 1939, p. 573.

[6] *Special Publication No. 145* (Manual of Second and Third Order Triangulation and Traverse), U. S. Coast and Geodetic Survey.

to 7".64. In the opinion of the writer these results are worthy of note, considering the program of observing used. Doubtless, the accuracy obtained is due primarily to two points given in the conclusion and emphasized herein: The use of high-quality instruments and the selection of the best qualified personnel. The writer doubts the advisability of attempting second-order and third-order triangulations by the program outlined in the paper, except for those surveying organizations best equipped in personnel and instruments.

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DISCUSSIONS

EFFECTIVE MOMENT OF INERTIA OF A RIVETED PLATE GIRDER

Discussion

BY MESSRS. WILLIAM R. OSGOOD, CLYDE T. MORRIS, B. R. LEFFLER, E. NEIL W. LANE, LEWIS E. MOORE, W. E. BLACK, AND L. E. GRINTER

WILLIAM R. OSGOOD,[4] M. AM. SOC. C. E. (by letter).[4a]—In discussing the interesting and valuable paper by Professors Lilly and Carpenter, it may not be out of place to call attention to a paper by Friedrich Hartmann.[5] This paper treats the question theoretically and discusses a test made in 1891 in the light of the theory. Professor Hartmann comes to the conclusion that in the absence of considerations of fatigue, girders of symmetrical cross sections may be designed safely on the basis of gross moment of inertia. In the case of an unsymmetrical cross section with a heavier compression flange than tension flange, however, deductions should be made for the rivet holes in the tension flange.

The authors state that loads causing flange stresses of 30 kips per sq in. gave no indication in the strain readings of permanent set. Was any such indication noted in the readings of deflection?

It seems quite possible, as the authors suggest, that the condition of the holes in the web and flange angles might account for the observed strain lines around these holes. It is likely that severe initial strains due to punching existed around these holes, and also, if the holes were not completely filled by the rivets, higher strains under load would be expected than otherwise. The strain lines might appear in the upstanding legs of the flange angles and not in the outstanding legs because the latter were connected to plates with drilled holes. The drilled holes would probably tend to a less erratic variation of strain in the plates, and this tendency would affect the strain in the connected

NOTE.—This paper by Scott B. Lilly, M. Am. Soc. C. E., and Samuel T. Carpenter, Assoc. M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[4] Materials Engr., National Bureau of Standards, Washington, D. C.

[4a] Received by the Secretary, October 25, 1939.

[5] "Sollen genietete Träger mit Berücksichtigung der Nietschwächung berechnet werden?" von Friedrich Hartmann, *Der Stahlbau*, Vol. 12, February 17, 1939, p. 25.

legs of the angles in the same way since strains in adjacent connected parts cannot be wholly independent of each other.

The authors state that it was their intent to obtain knowledge of the action of a plate girder under working conditions. Knowledge of the maximum load a girder can carry and the type of failure, particularly the behavior of the compression flange, is also so important that it would have been fortunate if this additional information could have been obtained for the girders tested.

CLYDE T. MORRIS,[6] M. AM. SOC. C. E. (by letter).[6a]—Professors Lilly and Carpenter are to be commended for the care with which their investigations were conducted and the completeness with which they have published the data secured. Experiments of this kind are important in extending the knowledge of strength of materials and confirming practical methods of design.

Presumably, the observed fiber stresses given in Column (6), Table 3, were computed from strain gage readings and, therefore, represent the average fiber stress over the 10-in. gage length. For girders PT-1 the holes in the outstanding legs and cover plates were spaced 5 in. apart, and therefore two holes would be included in the 10-in. gage length. Counting the holes as $\frac{11}{16}$ in. square (instead of round), which is on the safe side, the computed average fiber stress is $\frac{1}{10}\left[2 \times \frac{11}{16} \times \text{(Net I) stress} + 8\frac{5}{8} \times \text{(Gross I) stress} \right]$. For girders PT-2 the holes were spaced $2\frac{1}{2}$ in. apart and the computed average fiber stress is $\frac{1}{10}\left[4 \times \frac{11}{16} \times \text{(Net I) stress} + 7\frac{1}{4} \times \text{(Gross I) stress} \right]$.

TABLE 5.—STRESSES IN BOTTOM FLANGES

Girder	From Table 3				Computed in This Discussion	
	Column (10) I_G	Column (14) I_N	Observed Column (6)		PT-1 5-in. pitch	PT-2 2½-in. pitch
			PT-1	PT-2		
B-1	17.2	19.5	19.1	19.8	17.52	17.83
B-2	18.4	20.8	19.7	20.9	18.73	19.06
B-3	19.1	21.5	20.2	21.0	19.43	19.76
C-1	17.2	19.5	18.2	19.2	17.52	17.83
C-2	18.4	20.8	19.6	20.3	18.73	19.06
C-3	19.1	21.5	19.9	20.9	19.43	19.76

Table 5 gives the stresses in the bottom flanges taken from Table 3 and as computed by the method just given. In every case the observed average stresses given in Column (6), Table 3, are greater than the computed stresses using the foregoing method, and are only slightly smaller than the stresses using the net moment of inertia.

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[6a] Received by the Secretary November 3, 1939.

From these tests the writer would draw the conclusion that the customary method of design, using the moment of inertia of the net section with the neutral axis at the center of gravity of the gross area (Column (14)), gives results only slightly on the side of safety, and that the use of the moment of inertia of the gross section (Column (10)), as recommended by the authors, is dangerous.

B. R. LEFFLER,[7] M. AM. SOC. C. E. (by letter).[7a]—Conclusion 5 of this paper—that for ordinary girders, under usual loadings, the tests indicate the acceptability of the gross moment of inertia for design purposes—is fallacious, and the writer will attempt to show where the false reasoning occurs.

For many years structural engineers have known that the calculated deflection of the ordinary plate girder, using the moment of inertia of the gross section, gives results which correspond closely to the results found by testing. The false reasoning lies in assuming that deflection is a true indication of stress in a riveted girder.

To understand the meaning of what follows, it should be remembered that the deflection of a plate girder is caused principally by the shortening of the compression flange, and the lengthening of the tension flange.

To show the fallacy more clearly, consider a bar, 100 ft long, 6 in. wide, and 0.5 in. thick, with a 1-in. hole at the middle and on the axis of the bar. If the bar were stressed to a given amount, a certain total stretch of the bar would be obtained. Now, if a similar bar of the same quality of metal, without a hole in it, were stressed to the same amount, the total stretch of this bar would not differ appreciably from that obtained for the bar with a hole in it; the longer the two bars, the closer would be the agreement in the total stretching. Consider the bars as reduced to a length of 1 ft, the hole still remaining in one of the bars: It is probable that the total stretch in the two short bars would differ considerably.

If the investigator confines himself to bars 100 ft long, he would be justified, apparently, in concluding that a hole in a tension bar does not weaken the bar. This reasoning is entirely as valid as the reasoning leading to Conclusion 5. In fact, the writer regards most static tests on riveted structural parts to be of little value as a guide to correct designing, particularly if most of the stress is caused by a live load.

The fact is that engineers have not yet learned how to design, correctly, a simple tension bar with one hole in it when such a bar is subject to frequent live loads. Even a tension bar without a hole cannot have a uniform distribution of stress, for the reason that any rolled material is subject to surface defects which act as stress raisers. About all that is known about a bar under tension is the amount of average stress across the bar.[8,9]

Had the authors subjected the plate girder to an enormous number of repetitions of live load, they probably would not have reached Conclusion 5.

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[7] Engr. of Bridges, N. Y. C. R. R. Line West of Buffalo, Cleveland, Ohio.

[7a] Received by the Secretary November 6, 1939.

[8] "Tension Tests of Large Riveted Joints," discussion by Wilbur M. Wilson, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., October, 1939, p. 1452.

[9] "Fatigue Tests of Riveted Joints," by Wilbur M. Wilson and Frank P. Thomas, Eng. Experiment Station, *Bulletin No. 302*, Univ. of Illinois, 1938.

The ultimate average unit stress in any tension member can be determined only by the use of repeated loads, varying through a wide range of values. For a polished small specimen, the average unit stress will approximate the actual maximum unit stress; but such polished material is never used in actual structures.

The value of the average unit stress is a function of the range of stress and the number of applications. For a standard, Messrs. Wilson and Thomas[9] selected 2,000,000 applications. The average unit stress which produces failure after 2,000,000 applications is the ultimate strength of the material.

The authors have placed too much emphasis on the experimental location of the neutral axis. The location of the neutral axis can be varied over quite a wide range without materially affecting the results of calculation of stress in a plate girder, particularly in a deep girder. The internal resisting moment must equal the external moment. This internal resisting moment is a couple, the arm of the couple being the distance between the centers of gravity of the compressive stresses and the tensile stresses. The neutral axis is nothing but a dividing point between the compressive stresses and the tensile stresses; and this point is at the apexes of two triangles, granting the usual theory of flexure of a bent beam. Hence, a considerable variation in the location of the neutral axis can be made without affecting, to a great extent, the arm of the internal couple. It is to be remembered, also, that the sum of the internal tensile stresses must equal the sum of the internal compressive stresses, and this fact alone is of much more importance than the exact location of the neutral axis.

The paper may lead to erroneous specifications. The desire to save metal and simplify the designing of a riveted plate girder is a strong inducement to consider Conclusion 5 as valid.

E. Neil W. Lane,[10] Jun. Am. Soc. C. E. (by letter).[10a]—The importance of such basic experimental evidence as that supplied by the authors is seldom appreciated properly. Engineers, like men in other scientific groups in the search for truth, delve into detail to such an extent that sight is often lost of the major theme. Naturally this lays a weaker foundation in the form of an increased number of assumptions necessary to proceed with further detail. Messrs. Lilly and Carpenter have done a good job of attracting the attention of the profession to a major theme of structural design.

There has been considerable argument as to the relative merits of designing plate girders on the basis of the gross or net moment of inertia; and the younger, more inexperienced, and generally theoretical minded portion of the profession have tended to favor the more complicated and less accurate design procedure.

It is more or less common among structural engineers to use the gross moment of inertia in computing deflections because it will generally give a more reliable result than will the use of the net value. The authors have proved the experience that neither is exactly correct and that the users of the gross moment of inertia are nearly right. The experience of field engineers

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[10a] Received by the Secretary November 6, 1939.

with full-sized girders under actual working conditions serves as further corroboration that the gross moment of inertia is nearly correct within the elastic or design range.

Use of the net moment of inertia has led to additional safety at the expense of appreciable economy, a false idea as to design accuracy, and a misrepresentation of the stress distribution.

The fabricating methods now in use are the best that can be obtained at a reasonable cost. For this reason, riveted plate girder flanges will always consist of many small areas (adjacent to the rivets) of high stress restrained and stiffened by surrounding areas of low stress. It should be emphasized that use of the girder causes a plastic flow of the metal at points of extreme stress that makes the girder more capable of withstanding repeated loading. Drilling of members in place and the use of turned bolts is a dream for any but unusual conditions. Welding, although a step in the right direction for continuity and distribution of stresses, has evils of its own that limit its use.

The authors are to be commended for including the effect of diameter and pitch of flange rivets in their proposed formula for the effective moment of inertia. However, the writer would like to see the authors' studies include the effect of various ratios of fabrication—such as, depth to span length, web thickness to depth, and flange area to web area.

LEWIS E. MOORE,[11] M. AM. SOC. C. E. (by letter).[11a]—The experimental determinations were made by measurements of deflection under varying conditions and certain conclusions are drawn, particularly with regard to design, that seem to call for discussion.

According to the laws of mechanics, deflections and flange stresses in a beam are interdependent, and, in general, therefore, the deflections should be in proportion to the average flange stresses. By average flange stress is meant the average, considering not only gross and net areas, but the proportion of length which each governs; in other words, it must be a weighted average.

Considering the ordinary plate girder, the net area of a flange governs only on a plane perpendicular to the flange and passing through the centers of the rivet holes. If this plane is pushed parallel to itself along the axis of the flange the net area will increase until, when the plane becomes tangent to the rivet holes, it becomes equal to the gross area. This gross area will govern until the plane becomes tangent to the next set of rivet holes. A simple method of obtaining the average flange area is to compute the volume of the flange between successive clusters of rivets, less the volume of the rivet holes. Using the girder of the experiments, and omitting the web, the 5-in., flange rivet spacing yields the following:

Description	Area, in square inches
2, 2½-in. by 2½-in. by ¼-in. angles	2.38
1, 6-in. by ¼-in. plate	1.50
1, 8-in. by ¼-in. plate	2.00
Gross area	5.88

[11] Cons. Engr., Boston, Mass.

[11a] Received by the Secretary November 6, 1939.

The gross volume, center to center of rivet holes, is 5.88 in. by 5 in. = 29.40 cu in. The area of an $\frac{11}{16}$-in. rivet hole is 0.37 sq in., and the total length of holes is ($\frac{3}{4}$ in. by 2) + ($\frac{1}{4}$ in. by 2) = 2 in., making the volume of one set of rivet holes equal to 0.37 sq in. by 2 or 0.74 cu in. Subtracting this from 29.40 gives 28.66 cu in. net volume of steel between rivet holes.

The average flange area for the 5-in. length then is $\frac{28.66}{5} = 5.732$ sq in. The moment of inertia of the cross section will vary directly with the flange area. The deflection varies inversely as the moment of inertia. The ratio of the effective moment of inertia to the gross moment of inertia is then determinable from the average flange area, and theoretically, in the case in question, should be equal to $\frac{5.73}{5.88}$ or 0.975. This corresponds to the quantity "m" given by the author. For the particular experiment D-3 with dimensions of riveting as in the foregoing on the girder, "m" is given as 0.98.

This is a close agreement between theory and experiment and, incidentally, is a confirmation of the ideas of those who have believed it correct to compute deflections from gross moments of inertia.

The authors' conclusions regarding the position of the neutral axis agree with an opinion expressed by the writer in 1913.[12] The reasoning then was that the net section exists over only a short proportion of the beam and that between rivet holes the position of the neutral axis is certainly found by using the gross area. It seems entirely reasonable to assume that the neutral axis does not shift up and down at every rivet hole, but remains in substantially the same vertical position throughout the length of a properly designed beam. It seems reasonable, furthermore, to assume that this position will be nearer to the neutral axis of the preponderating section, which is the gross section.

The question of the propriety of using the gross moment of inertia or the effective moment of inertia for designing is a quite different one.

The author states that conditions of stress on net areas through rivet holes are variable, and that the metal is stressed, in part, beyond the elastic limit at these locations, using customary working stresses.

As a simple illustration of what happens, consider the application to the experimental girder. Suppose the effective moment-of-inertia method is adopted, with the value determined by the experiment, which is 0.98 of the gross, and only the flange is used. With a design stress of 18,000 lb per sq in. one should use 18,000 × 0.98 = 17,640 lb per sq in. to calculate the gross moment of inertia, and to proportion the flange. The net area (with $\frac{11}{16}$-in. holes) of the flange is 5.88 − 1.02 = 4.86 sq in. The net area is then $\frac{4.86}{5.88}$ × 100 = 83.0% of the gross. This means that the average stress on the net area is $\frac{17,640}{0.830}$ = 21,200 lb per sq in., or about 18.0% above the allowable of 18,000. In view of the fact that there is some unavoidable overstress at the rivet holes when designing in the usual way with usual stresses, it is doubtful if the effective moment of inertia method is proper to use when designing.

[12] "Design of Plate Girders," by Lewis E. Moore, 1913, p. 42.

For quick designing by the section-modulus method the writer has found that, for ordinary plate girders, it is sufficient to determine the section by applying 85% of the allowable tensile stress to the moment and then using the tables for moments of inertia of gross sections. The designs so determined rarely need much revision of the main sections.

W. E. BLACK,[13] JUN. AM. SOC. C. E. (by letter).[13a]—While engaged in testing one-quarter size steel rigid frames at Lehigh University at Bethlehem, Pa., in 1938, the writer had occasion to investigate the effective moment of inertia that was being obtained in the tests. For one test specimen, a length of girder having constant cross section was tested as a simple beam loaded symmetrically with two equal concentrated loads. In the part of the beam having maximum moment, top and bottom flange strains were observed at various points, and the extreme fiber stresses were computed from them. The effective moment of inertia, as computed from these stresses, was found to be less than 3% smaller than the gross moment of inertia, and comparatively greater than the net moment of inertia. The cross section of the model girder at the location being tested was 6 in. deep, and consisted of a 6-in. by $\frac{5}{32}$-in. web plate and four 2-in. by 2-in. by $\frac{1}{4}$-in. angles, riveted together by 0.5-in. rivets having a pitch of 3 in. This section, although small, represents a girder of decidedly different proportions than that tested by the authors, and it is gratifying that results obtained in the two cases were similar.

The method of computing the effective moment of inertia used in connection with the aforementioned rigid frame tests was based upon an empirical relation that has been used previously by some designers. In this method, the volume of metal removed by the rivet holes in a given rivet line is considered to be uniformly distributed along the length of the girder at the same distance from centroidal axis as the rivet line. Thus, the reduction in moment of inertia of a given section due to one line of rivets would be, approximately:

$$\Delta I = \frac{A_h g}{p} d^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which ΔI = reduction in moment of inertia due to the rivet line; A_h = cross-sectional area of rivet hole; g = grip of rivets; p = pitch of rivets; and d = distance from centroidal axis to center of rivet line.

In applying this relation, the neutral axis was assumed to remain at the centroidal axis of the gross section and the deductions were made from top and bottom flanges alike. This computed effective, or average, moment of inertia gave agreement within 1% of the experimental value of moment of inertia determined by the foregoing test.

To test the applicability of this simple method further, the writer prepared Table 6, which is based on the authors' tests. The experimental values are taken directly from the data presented in the paper. The computed values of $m = \frac{I_E}{I_G}$ (see Table 4 and accompanying explanation in the paper) and of the

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[13a] Received by the Secretary November 6, 1939.

flange stresses were calculated using the effective, or average, moment of inertia just described. It will be noted that the comparisons are made for only the bolted and riveted sections, which may be expected to simulate an actual plate girder in behavior. The comparisons show that the computed average

TABLE 6.—EFFECTIVE MOMENT OF INERTIA

Section	Comparison of Values of $m = \frac{I_E}{I_G}$				Comparison of Flange Stresses—Stresses in Kips per Square Inch					
	Specimen PT-1 P = 5 In.		Specimen PT-2 P = 2½ In.		Specimen PT-1 P = 5 In.			Specimen PT-2 P = 2½ In.		
	By experiment	Computed, using average I	By experiment	Computed, using average I	By Experiment		Computed, using average I	By Experiment		Computed, using average I
					Top flange	Bottom flange		Top flange	Bottom flange	
C-1	0.970	0.972	0.928	0.943	17.7	18.2	17.7	19.0	19.2	18.2
C-2	0.970	0.974	0.936	0.945	18.9	19.6	18.9	19.4	20.3	19.4
C-3	0.962	0.975	0.916	0.953	19.7	19.9	19.6	20.0	20.9	20.0
C-4	....	....	0.930	0.953	....	....	....	....	....	....
D-3	0.980	0.975	....	....	19.3	20.2	19.6	20.3	20.3	20.0

moment of inertia falls within approximately 1% of the experimental moment of inertia for test section PT-1, having a rivet pitch of 5 in., and within 4% for test section PT-2, having a rivet pitch of 2.5 in. The observed and computed values of flange stresses have a maximum divergence of 5% for any given section. It will be noted that the agreement obtained for the riveted sections is considerably closer than for some of the bolted sections.

The writer does not offer this method for determining the effective moment of inertia of a girder section in place of the authors' or any other theoretically sound formula, but rather to supplement such formulas as a simple working relation that will achieve approximately the same results. It is of interest to note that the values of $m = \frac{I_E}{I_G}$ computed by this method agree very closely with those computed by Equation (5), for $k = 0.6$. Regarding the application of any method for determining effective moments of inertia, the writer concurs with the authors that in the ordinary case, gross section may be used for design without fear of serious error. In those instances where the amount of material removed from a plate girder by rivet holes is unusually large, the test data indicate that an effective moment of inertia should be used. The method of determining the effective moment of inertia to be used in such cases requires, for complete justification, further tests on girders which have greater percentages of material removed by rivet holes, and from both the web and the flanges.

Many engineers, admitting the advisability of using an effective moment of inertia for the computation of girder deflections, still insist upon a net section when computing stresses. There is some basis for this stand in that the actual stresses existing adjacent to a rivet hole are probably much greater even than

those computed by the use of net section. Were the structural failure of a plate girder to occur by rupture across a transverse row of rivets, the use of net section would be justified. However, since the structural failure of a plate girder, under static loads, will occur, not by rupture, but by the general yielding of a comparatively large portion of the girder near the location of maximum moment, the high local stresses near the rivet holes and the high average stress across a transverse row of rivets lose much of their significance. Many tests have borne out this predication. Therefore, the observed flange stresses obtained by the authors using a 10-in. gage length, although not the actual maximum stresses, are the significant stresses when the prospective failure of the girder is being considered. The agreement between the observed flange stresses and those computed using the observed effective moment of inertia (see Table 3) is evidence enough that the effective moment of inertia may be used for computing stresses.

L. E. GRINTER,[14] M. AM. SOC. C. E. (by letter).[14a]—The profession is indebted to the authors for prompt publication of these tests on plate girders. Recent changes in design specifications preceding the publication of these tests but possibly to be justified thereby, have upset standard procedures in plate-girder design and, naturally, have disturbed conservative designers. Net section, as well as its importance in design, has been taught to engineering students since the beginning of engineering education, and its sudden elimination from certain accepted specifications was too startling to be accepted without question. It is evident, therefore, that these tests, which are considered practically to justify the use of the gross moment of inertia, will be inspected with the greatest care.

It is first noted that the two girders tested were only 12 in. deep. Hence, even the theoretical displacement of the neutral axis was a fraction of an inch, and the observed displacement was too small to be measured consistently by the available measuring devices. Another objection to a shallow girder as a built-up member for testing purposes is that the strains introduced by fabrication would produce a disproportionate influence upon the test results. The irregularities in Table I would undoubtedly have appeared far less serious if the girder had been several times as deep. For instance, rivet slip, if it had occurred, would represent a serious influence upon the test results for these shallow girders.

Neutral Axis.—Consistency is evident in the test results, reported in Table 1, in that all data indicate a small shift of the neutral axis in one direction. This shows the care with which the tests were made and confirms clearly the long-held belief that some deduction from the gross section is necessary to allow for the influence of holes whether open or filled. The fact that strain measurements were made on a 10-in. gage length which spanned from two to four rivet spacings indicates that the authors were not attempting to do more than determine an average eccentricity of the neutral axis. The second important obser-

[14] Vice-Pres., Armour Inst. of Technology, Chicago, Ill.

[14a] Received by the Secretary November 10, 1939.

vation from Table I is that the location of the neutral axis, as obtained from flange measurements, does not agree with the location as found from web strains. Since these strains were obtained from a section where the shear was zero, one can only conclude that there must have been some rivet slip, or the theory of "plane sections" failed to apply for some other unknown reason. This is rather disturbing if conclusions are to be drawn from these data.

If one notices that the average observed displacement of the neutral axis for all recorded data on specimen PT-1 is about one sixth or one seventh of the computed displacement, an important observation can be made. This specimen had $\frac{11}{16}$-in. holes at 5-in. spacing. Hence, less than one sixth of the length of the girder was influenced by the net section, and the observed "average" displacement of the neutral axis was of the magnitude that should have been expected. Whether the neutral axis actually shifts upward at each hole in the flange is questionable, perhaps, but the influence in added flange stress must be considered in design until it is proved non-existent. Such proof is still lacking. Fig. 7 shows, pictorially, the possible shift in the neutral axis above a flange hole. Proof of this action is also non-existent at the present time.

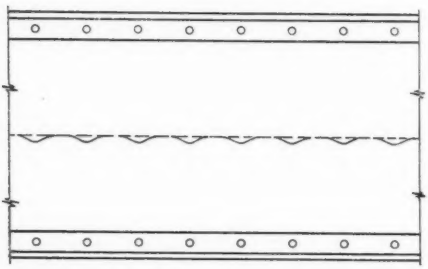

FIG. 7.—POSSIBLE UNDULATIONS OF NEUTRAL AXIS

Effective Moments of Inertia.—The method used to measure the effective moment of inertia was based upon the observation of deflection. Of course, this indicated, accurately, the proper moments of inertia for use in future deflection calculations, but it scarcely seems significant for maximum stress determination. A deflection is produced by the cumulative influence of the stress at all cross sections. Since a major part of the length of the girder functions as a solid cross section without holes, it is self-evident that the effective moment of inertia, when determined from measured deflections, will represent an "average" moment of inertia which will approach the gross moment of inertia closely. Surely there can be little connection between this "average" moment of inertia and the proper value for use in the flexure formula.

Flange Stress.—Again dependence is placed upon a 10-in. strain gage and recorded values are carefully determined averages of the readings taken from four to eight gage lines. Recorded values, therefore, do not represent even the greatest "average" strains for 10-in. gage lengths. What maximum strain would have been recorded on a shorter gage length is purely speculative. Naturally, it would have been higher than the "average" values recorded, which are from 2,000 to 4,500 lb per sq in. lower than computed values from net moment of inertia. It is clearly within the range of probability that strains could have been measured on $\frac{1}{2}$-in. gage lengths quite as high as the theoretical values. In fact, considering possible torsion in the cross section, it would not be particularly surprising to find measured strains that exceeded the theoretical strains for gage lengths of 1 in. or less.

Stress Concentration.—There has been much confusion about the subject of stress concentrations, and discussions of rivet-hole stresses have not been free of such influence. There are high stresses around punched rivet holes, and there would be high stresses even if plastic distortion had not taken place during punching. However, residual stresses from temperature change, or from overstrain, and even stress concentrations from discontinuities, are unable to resist load. A tension member with a hole may have a high localized stress, but this stress does not increase the load capacity (in fact, it reduces it) because the average stress over the net area A_n is still $\frac{P}{A_n}$. In exactly the same manner the stress concentrations around the rivet holes of a plate girder do not produce moment resistance and, in fact, they are more related to fabrication methods and similar influences than to the applied moment.

Perhaps a clearer statement of the action of stress concentrations, residual stresses, etc., is that they are balanced stresses—that is, tension and compression occurring together. Thus under the smallest plastic flow they begin to reduce, and eventually they disappear altogether. Such stresses are not to be neglected lightly, but they are obviously less likely to produce failure than direct stress and flexure where direct load resistance is involved. Statics requires the average stress on the net section of a tension member to be equal to the load divided by the net area. The laws of statics permit no loose thinking in this matter, and it would be a very incautious investigator who questioned this fact or permitted test data to change his thinking in this regard. Perhaps bolts could be tightened through holes in a tension member with sufficient pressure to reduce the net effect of the holes, but engineers are not likely to be willing to let this possibility influence their belief that tension members must be designed for net section.

The lower flange of a girder is as much a tension member as is a chord eyebar of a truss. In fact, it serves the same purpose. Apparently any argument about stress concentration resisting load that can be applied to one can be applied to the other. Any thought that stress concentration over the entire section of the tension flange which is an increase of "average stress" can be neglected in the same way that residual stress and stress concentration around rivet holes are neglected is subject to serious criticism since it conflicts with the requirements of statics. Admittedly, this increase of stress over the section of the tension flange at a rivet hole may exist over a very short length of the flange and may scarcely be found at all when measurements are made over a 10-in. gage length. However, like a weak link in a chain, the effect upon the whole is serious, as statics demonstrates unerringly.

Further Tests.—It is not beyond the reach of modern testing methods to determine accurately the influence of rivet holes upon "total" or "maximum" flange stresses as distinguished from the authors' determination of "average" flange stresses over a 10-in. length of span. With sensitive gages, strains could be measured on $\frac{1}{2}$-in. gage lengths at a dozen or more positions across one section of the web and the tension flange in line with a rivet hole. It would be preferable to perform this test under light loading so that the problem of inelastic strain near rivet holes would not be involved. Then, by transferring

unit strain into unit stress and by summing the total stress over the flange and web section, it would be possible to check either the so-called approximate or exact methods of plate-girder design. Such a test, to be most useful, should be made on a test girder of reasonable depth, as representative as possible of standard construction. Of course, there could be no particularly useful design information obtained from single deflection measurements because deflection is a measure of average resistance, whereas the interest in the present case is in the resistance of a particular section. The final check on the accuracy of such testing is obtained from statics. The total moment resistance of the section obtained as the summation of the moment resistances of its fibers must equal the applied bending moment.

Specifications.—Specifications should follow, and not lead, theory. Testing a procedure by introducing it into specifications before it has been checked by statics, well substantiated by tests, and even by use, is dangerous from more than one point of view. The questionable use of the gross moment of inertia in plate-girder design is a case in point. The influence upon the cross section is so minor that it is not likely to endanger a structure physically, but it seems to the writer that its introduction into specifications was far more serious, in that it tended to develop mental carelessness. The laws of statics are fixed matters that must not be changed by specification writers. When working stresses are to be increased, they should be straightforwardly raised, but "design moments" which are controlled by statics should not be reduced even if the influence upon the structure is the same. The profession suffered from the use of that artifice in flat-slab specifications, and there is no reason to reestablish a similar practice with reference to plate girders. The next step would be to omit the consideration of net section in tension members. Of course, this is no criticism of the authors' tests, which were undoubtedly well planned and carefully performed. The criticism is of premature use of anticipated results by specification writers.

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